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Earth Dam Construction

ONE of the features of the Seros hydro-electric development is the careful way in which the earth dams have been designed and the materials for them specified. From the article by J. C. Stevens, on page 250, it is apparent that the perviousness of the materials increases witn the distance from the upstream slope so as to lessen the resistance to possible seepage through the dam. This feature, in connection with a rock dike near the downstream toe, will determine the hydraulic gradient through the dam and prevent it from intersecting the downstream slope at or above ground level.

Emergency Shipping Measures

CONSISTENT progress toward the provision of adequate shipping facilities between North and South American ports has marked the past week. Hard upon the heels of the new registry act came the war-risk insurance bill. A measure authorizing the Government to purchase ships through majority stock ownership of a shipping corporation has been introduced in the Lower House and relief from present requirements as to crews is almost certain. So far there has been nothing but commendation of the course pursued by the Government, though there is some opposition to the Government's ownership of merchant vessels. The emergency, however, is a great one, and if private capital will not come forward the Government must take the necessary steps.

Dependence on Internal Resources

AS one of the favorable effects of the war in this country we may expect still greater appreciation than in the past of the magnitude and variety of our internal resources. Many of these have lacked vigorous development because the cost of production, in the development stages, was considerably higher than that in the matured competing industries abroad. In the present juncture there is certain to be a revival of interest in these products. Ball bearings, for example, have been produced in considerable quantities in the United States, but industries which use them in their products have, nevertheless, been dependent in a large measure upon European production. These supplies now being cut off, the Hess-Bright Manufacturing Company, of Philadelphia, is proceeding immediately with the erection of an additional factory to supply the American demand. While some industries cannot be established here in the comparatively short length of time that the European nations will probably be at war, there are doubtless many that can be rapidly equipped and that can establish a trade before European competitors are again on the commercial firing line.

Withstanding the Shock

IT is nothing short of inspiring to read the comments of the British engineering and trade journals on the present European war. Refraining, naturally, from a discussion of the ethics of the situation, they have devoted themselves to a careful study of industrial conditions and have endeavored, with success, to point out reasons why business should continue in the customary channels. The "Electrician," of London, has expressed itself particularly well, and has summarized the attitude of manufacturers as determined by answers to the queries of staff representatives. In the issue of Aug. 14 it gives a most encouraging view of the situation. So far there has been little or no dropping off in the industry which it represents, while the cutting off of shipments from continental houses has greatly stimulated certain lines, fully making up for the drop in exports in other branches of the trade. It suggests that each dealer in electrical supplies post in his window a sign marked "Business as Usual," which is the British equivalent of our expression, "Business Going on as Usual," customarily posted when alterations or repairs are in progress. American manufacturers who are inclined to be panic stricken at the present time may well ponder the British attitude and the actual observations of the British journal. Certainly our position is immeasurably more favorable than that of England. There is temporary inconvenience here, but if English business has suffered but slightly, we need not fear for ours if we but keep our heads. We, too, may well post the sign, "Business as Usual."

Waterway Terminals

IT is well known that transportation on our inland waterways suffers more from lack of proper terminal facilities than from inadequate water depths. In other words, we are nowhere near the limit of carrying capacity for either the Ohio or the Mississippi as far as depth is concerned, the limitation at present being the lack of proper wharfage facilities at the small towns. In this respect Europe is far ahead of us. Evidence is at hand, however, to indicate that reasonable progress is being made toward supplying the needed facilities. In the issue of this journal of Aug. 8 the municipal terminal at Kansas City was described and in this issue the terminal engineer of the New York State Barge Canal describes typical structures on that waterway. While he does not enter into a discussion of the economic and transportation phases, his introductory paragraph is a text upon which an elaborate sermon might be preached. There is no doubt that the new canal will not suffer the decadence of its predecessor on account of lack of proper dockage facilities. Much remains to be told about these interesting installations in addition to what Mr. O'Connor has described in his article, but the further data will not be available until the warehouses and freight-handling machinery have been designed. Then another interval should make available some interesting operating data that may afford a new stimulus for proper docking facilities on all of our navigable internal waterways.

New England Waterworks Convention

FROM the announcement of the program for the thirty-third annual convention of the New England Waterworks Association at Boston next month it is apparent that the committee has made a particular effort to serve the interests of the waterworks superintendent. Two entire sessions are to be devoted to the discussion of the, so-called, practical problems of management. This policy will be approved by most of the members. While the highly technical paper—dealing, for example, with formulæ for the flow of water in pipes—has its place at any waterworks meeting, a judicious allotment of time between theory and practice is more desirable than an overbalancing of highly technical information. It is interesting also to learn from the program that the committee on filter operating statistics—the first of its kind to be created by any waterworks association in this country—will submit its first report. There is at present nothing which approaches uniformity in methods of reporting filter operating results. If the work establishes uniformity the profession will again be indebted to the association.

Doing Good Road Work

ONE of the most conscientious and best-equipped road contractors operating on Long Island recently exclaimed in disgust, "I am almost tired of trying to do good work!" He did not mean it but expressed a state of mind resulting from the fact that an award had just been made on the basis of $1.14 a square yard for some work on which the "good contractor" had bid $1.16. The man who got the work can not even read the specifications intelligently. He has no adequate organization or equipment, nor a staff skilled in road construction. The work will cost him more than he figured and the temptation to "skin" the job will be very strong. But even if he resists this and does the best work he knows how to do the county will not get as good a road as it could have bought for two cents more per square yard. The well-equipped contractor has—besides his plants, capital, organization and reputation—a big mileage of the best roads in the community to show what he can do.

Much might be said about the unwisdom of foregoing all these advantages for two cents a yard. The point here emphasized, however, is that the good contractor should not follow the unwise policy of officials. The time is approaching—in fact it has arrived in many places—when communities will pay a little more for good work by responsible contractors. This is but a logical extension of the rule in other branches of contracting, in which not the lowest but the best bid gets the contract. Good contractors are so close to their reward that it will be a loss to themselves as well as to the community if they grow weary in good roadbuilding.

Municipal Electrical Inspection and the Fire Hazard

ELECTRICAL inspection by municipalities progresses from year to year with so little blowing of trumpets that few people outside the field realize the importance it has attained or the beneficial results of such service when conscientiously performed. In the city of Chicago alone the inspection bureau dealing with electrical matters has no less than forty-five outside men, more than half of the staff being in the field. The taxpayers are contributing $124,000 this year to secure safe electrical installations. Nearly 115,000 inspections were made in 1913. Since 1894 the number of electrical fires in Chicago per year has increased only 10 per cent, although the connected load in incandescent lamps has increased 1400 per cent. Less than two electrical fires per week occur in this city of nearly two and a half million inhabitants. Improved electrical design and construction, of course, share in the credit for the foregoing excellent record, as well as inspection.

Broader responsibilities confront the municipal electrical inspector than formerly through the multiplication of moving-picture theaters, the extension of outside lines in residential districts, popularization of electrical conveniences and the necessity of reinspecting old installations. The last problem is an important one and the outlay for it is well spent. It is pleasing to note that both the public and the contractors are becoming more appreciative of the inspection bureau's efforts and that early hostilities and attempts to "put things over" at the expense of the inspector are steadily decreasing. Many large industrial plants have found that they save money by living up to the wiring code and trying to get the utmost benefit out of the inspector's advice, which, of course, often carries beyond his strict municipal orders.

Troublesome problems still confront these departments in many cities, notably those associated with the irresponsible purchase of inferior apparatus by consumers, the intentional overloading of circuits and the use of fuses of unwarranted capacity on temporary jobs. Contractors using temporary power installations can improve their distribution facilities in many cases at small expense and in the interests of safer operation. Certainly every user of electrical service who co-operates with the generally competent and helpful inspection bureau force is taking the wise course. Reduction in fire risk will surely ensue.

Preventive Maintenance

THERE are two kinds of maintenance practised by street and road departments—one very common and one very unusual. The common kind of maintenance is really nothing else but repair work made necessary by the condition of the roadway. In a few places these repairs are trifling at each spot mended because the work is done as soon as weakness has resulted in minor defects. It may or may not prove more economical, year in and year out, than the method of waiting until the street is full of holes and ruts before repairs are made, but it keeps the roadway in much better condition for use. It is a pity that repair and travel statistics are kept in such a way that it is usually impossible to determine whether continuous or periodic repairs are more economical on a road, because before long this subject will be one which will cause very earnest thought in many cities. The annual cost of good streets and roads is not yet appreciated in most of our cities, for we have been neglecting maintenance and shoving first charges off on posterity through the legerdemain of long-term bonds, corporate stock and such subterfuges for legitimate financing. This actual annual cost is very largely a matter of maintenance, and it is remarkable that so little thought is given to what Clifford Richardson has aptly termed "preventive maintenance."

What is meant by that expression can be made clear by an illustration familiar to every owner of a house. If it is allowed to go without repainting until the sills are covered with a dead coating and the paint on the walls shows flakes and bare spots, the wood will have deteriorated in many places. If the paint is renewed as soon as it loses its life, the wood is preserved and the house is kept in better condition with little or no more expense than is needed for good work at longer intervals.

It is much the same with a high-class pavement, made with a tar or asphalt binder. An expert can detect a deadness about such a pavement or road just before holes begin to develop in it. At such a time it is usually possible to give life to the pavement by coating it properly. The long life of the old Fifth Avenue pavement in New York, due to the oil dropped on its surface from automobiles, is an excellent illustration of how the natural term of service of an asphaltic surface can be prolonged. The detection of and remedy for incipient holes and ruts call for expert advice, and consequently there is not much probability that "preventive maintenance" will receive attention generally for a long time to come. But in view of the money we have already paid for better pavements, and particularly in view of the heavy load of good roads' debts we have put on the shoulders of the next generation by our financial methods, it would indeed be most satisfactory to see some of our cities adopt the plan. It is the system of maintenance that keeps the great liners in service between Europe and America, that has enabled the Manhattan Elevated Railway to make a wonderful record of freedom from mechanical mishaps and that keeps the pumping engines in a well-managed waterworks system working smoothly.

Irrigation Management as Against Irrigation Engineering

STRENUOUS efforts are being made by the United States Department of Agriculture to bring home to the American farmer the conception of his avocation as a business and not an isolated occupation to be pursued regardless of the rules laid down by long experience for success in other walks of life. A reading of the article by Mr. Newell in this issue of the Engineering Record will convince one that a similar propaganda is needed in order to emphasize the qualities and attainments necessary for the successful conduct of a completed irrigation system.

Irrigation work thus far, even to those directly concerned with it, has been an engineering problem. Two factors have been given inadequate consideration—the agricultural and the human. Only a brief survey of Western experience is needed to show how vital these are if the full possibilities of our arid West are to be developed. Where failures have been registered they are due not to engineering mistakes so often as to ignorance of the water and fertility requirements of crops and to neglect of the man himself. Fortunately, the importance of the agricultural and human considerations is now fully recognized and with that recognition there has returned confidence in the ultimate success of Western reclamation work, whether under public or private control.

It has been said that the production by one man of five times as many potatoes per acre as his neighbor rests on psychological grounds. Acquaintance with the method of thought of the average farmer is hardly necessary to convince one of the truth of this assertion. In fact, it is almost self-evident. The same remark could be applied to irrigation management. It is entirely a matter of psychology, and as such will be most successful when the manager is a man of broad sympathies, endowed with the ability to read human nature, strengthened by a strong sense of justice and mellowed by contact with suffering and hardship. For one thus equipped irrigation management holds forth remarkable opportunities for usefulness. Needless to say experience with both the agricultural and engineering sides of irrigation work is necessary. As always with the successful director, the irrigation executive is a leader as well as an executive. His project will reach its ultimate goal as much through the ideals he is enabled to hold up to his settlers and the advanced methods he can induce them to employ as through the routine handling of the storage and distribution works under his charge. Irrigation management is a work worthy of the mettle of strong, able men.

Engineer's Influence in Fixing Fire Insurance Rates

LAST spring the United States Supreme Court upheld the validity of the Kansas statute regulating fire insurance—a decision commented upon in this journal May 16, 1914. In the majority opinion handed down by Justice McKenna, the following statement is made: "The price of insurance is not fixed over the counters of the companies, by what Adam Smith called the higgling of the market, but forms in the councils of the underwriters, promulgated in schedules of practically controlling constancy, which the applicant for insurance is powerless to oppose, and which therefore has led to the assertion that the business of insurance is of monopolistic character and that it is illusory to speak of a liberty of contract." A dissenting opinion was filed by Chief Justice White, Justice Lamar and Justice Van Devanter.

As a result of the decision it is to be expected that all States will within a reasonable time have regulations governing fire insurance rates. In the formulation of these laws engineers, as representatives of water and of insurance companies, should have a prominent part. It cannot be denied that the underwriters have often been arbitrary, that the rates are often based on what the traffic can bear, and that instances of gross discrimination have been too common. It is therefore not strange that some of the States have passed very drastic laws. So many companies ceased writing fire insurance in Missouri and Kentucky when their laws went into effect that business was seriously inconvenienced, and it has been necessary in each State to suspend the laws to which the companies objected until the matter can be investigated by commissions, and until new codes can be adopted.

Small towns must charge more than three-quarters of the expenditure for a waterworks system to fire protection. In the larger cities from one-third to one-half of the cost must be so charged. Such expenditures should have decided effects upon insurance rates. For example, an average village of 1000 inhabitants probably has insurable property in excess of $200,000. Assuming that the average fire insurance rate is reduced from 80 cents to 50 cents per $100 per year after putting in a waterworks system, insurance premiums could be reduced about $600 per year, and to that extent offset the increased taxes, an argument which would probably increase the number of villages owning waterworks systems. However, at a recent meeting of waterworks men it was the consensus of opinion that it was useless to expect a material reduction in rates because of improved fire protection.

It is time that the making of a fire insurance rate should be put on a scientific basis, one that is based on loss experience under various degrees of protection. Since the decision of the Supreme Court the underwriters can no longer claim that theirs is a private business into which no one may inquire.

Aviation in the European War

ENGINEERS have been watching with great interest the first performances of the warring aviators in plane or dirigible. A great deal of a sensational nature has been predicted concerning them but thus far their rôle seems to be confined to scouting exploits of an extremely useful and most daring character. As weapons of offense they have yet to show any very substantial value, in spite of much promiscuous dropping of bombs already reported. Indeed anyone familiar with the properties of high explosives is not likely to attach much importance in the military sense to the dumping of any amount which can be carried in aviation even on a big Zeppelin. It is, of course, possible to do a certain limited amount of damage and produce a large moral effect by dropping bombs from the sky, but no fortified place is likely to be inconvenienced or damaged to any material extent by this means. Moreover, a soldiery already fairly inured to shell fire is not likely to be worried by a few casual bombs dropped by airships. Then, too, it has been already abundantly shown that airships of any kind flying low enough to direct a bomb attack to any advantage are extremely likely to be riddled and brought to earth.

Lists of casualties permitted to escape censorship in the present struggle are scarce and unreliable, but enough has transpired to make it very evident that both aeroplanes and dirigibles have suffered seriously from gun fire and that numerous casualties have resulted. Reports have reached the public of the destruction of at least half a dozen dirigibles and considerably more than this number of aeroplanes. The dirigible is particularly susceptible to attack by gun fire on account of its enormous size. The Zeppelins, for example, run from 450 to 500 ft. in length and from 45 to 50 ft. in diameter. Hitting a target of this size with a rapid fire gun at a distance of a mile or two is a task which does not worry the artillerist in the slightest degree, even when the ship is flying at its top speed of 40 to 50 mi. an hour. Once hit even the most effectively subdivided gas bag is not likely to stay long aloft, even if it escapes immediate destruction by fire or the wreckage of the delicate aluminum girders by which it is stiffened. The aeroplanes, from their much smaller dimensions and higher speed, are immensely harder to hit than the dirigibles, but persistent machine gun or volley firing is quite likely to disable the aviator or carry away some vital part of the machine if the flight is anywhere within rifle range. At greater distances a well-directed shrapnel fire is likely to prove effective. It is very questionable whether any of the dirigibles now in use can fly high enough for any considerable distance to be out of range of artillery fire.

So, despite the great usefulness of both classes of machines for scouting purposes, their offensive powers seem to have been considerably overestimated. Only in night attacks against places unprovided with searchlights and guns can they give much hope of inflicting even moderate injury, particularly since the propellers make a din audible for a long distance. Several ingenious efforts to create a preposterous scare on their account have already appeared in the public press, but no engineer will take them seriously.

As regards the aeroplanes the possibilities of mischief are not very serious. The dirigibles have a long radius of action and can carry a considerable quantity of explosives, but their capabilities in these respects are pretty well understood. There are no known non-burning gases possessing any considerable lifting power, which must always leave the dirigible extremely vulnerable, and there are no mysterious explosives capable of much greater destructive power than those already well known, for the character of high explosives is perfectly well understood by chemists and the improvements in later years have been almost wholly in the direction of greater safety, making practicable their use in shells. Doubtless in the present terrific struggle daring and sensational use will be made of air craft, but their offensive power is extremely unlikely to influence in any material way the outcome of the struggle.

Reports, therefore, of terrible raids about to be made by fleets of airships may be very heavily discounted, for certainly no nation possessing a fleet of airships would care to risk them in a raid which would certainly end in their destruction, probably before they were able to do any considerable damage. Aeroplanes might get through for a fairly long raid without being hit, but, on the other hand, the injury they can inflict is so very limited, owing to their carrying capacity, that no one need worry much about them. For attacking battleships dirigibles might carry enough explosives to inflict considerable damage to the superstructure if they could get near enough to land the charge successfully, which is highly improbable. The bombs dropped from aeroplanes would be extremely unlikely to do much mischief unless they dropped down a funnel which chanced to be unprotected with the gratings provided to block just such attempts. The defensive precautions against aerial attack are merely rapid-fire and machine guns on mounts permitting high elevation, and served by artillerists familiar with the trajectories of their pieces for high-angle fire. Such weapons are now plentiful even if they were not prepared at the beginning of the war, for the mounts present no mechanical difficulties, and necessary information for the artillerists could be plotted by any expert ordnance officer in half a day. A judiciously placed smoke-shell or two is sufficient to give the approximate range and then the destruction of the dirigible within the next five minutes is fairly well assured. It is all very well to proclaim airship raids in scare headlines as a ruse for terrifying nervous old ladies, but the real fighting forces by land or sea, equipped with searchlights and rapid-fire guns, are not likely to be seriously worried. For scouting purposes aircraft are invaluable and they may occasionally do some minor damage to the enemy, but their offensive powers are, in the nature of things, rather restricted.

The Human Side of Irrigation

The Manager of the Completed System Faces Problems as Complex as Those of Engineers Who Build the Dams and Canals

By F. H. NEWELL
Director of the United States Reclamation Service

A PROSPEROUS, intelligent and contented rural population is essential to our national perpetuity. This is the lesson which history is teaching in all parts of the world. The opportunities for such a population are widely scattered, but are found particularly attractive in the arid regions. Under conditions of good irrigation management it is possible to attain more nearly ideal conditions there than in the humid regions, where the farmer is more largely dependent on chance in the way of adequate rainfall. It is possible by good management in an irrigated country to increase the income of the farm from three to fivefold above that of similar land elsewhere—to double the crop on the acre while halving the cost. The basis of this better rural life is the greater earning capacity of the irrigator over that of the ordinary farmer.

The management of an irrigation system has for its purpose the delivery of water to agricultural lands at such times and in such quantities as will enable the irrigator to produce the largest and best crops. The success of the manager is measured largely by the success of the farmer. The proper management of irrigation works necessitates a skill and experience as complete as that required for a railroad system, city waterworks, or a gas and electric-light system. The irrigation manager should have had years of practical experience not only in the construction of the works, but more particularly in those matters which have to do with the distributing of water to the farmers and with the details of agricultural operations.

PROBLEMS OF MANAGEMENT

After an irrigation system is built, or when it reaches a degree of completion where water is available for any considerable area of the land, then begin the problems of management. These continue for generations, or as long as agriculture is practised by the use of these works. At first the difficulties are very great, but as time goes on certain practices become fixed and the vexing questions gradually subside into routine details.

At the outset the man in charge of the operation and maintenance of the completed portions of the irrigation system is confronted with a variety of problems in the solution of which there are no well-established precedents. He must trust largely to his own judgment and to such experience as he may have had, verified as far as possible by a knowledge of what has been done by other men. He needs suggestions, and to fill such need in part the following statements are prepared.

At the initiation of the undertaking the manager has not only his own problems of a new system and of establishing desirable precedents, but his work is greatly complicated by the fact that most of the water users are novices and wholly unskilled in irrigation. The land has not been leveled, nor made ready for economical irrigation; everyone concerned is working under the greatest of disadvantages. The irrigators as a body are not only inexperienced, but many of them are disappointed in that they have expected easier things. Thus they are often critical even of the details in which the manager is proceeding in the best possible way. There has been attracted to the locality an exceptionally large number of men who have never made a success elsewhere. These men attribute their failure to make good under the new conditions, not to their own inability but largely to the faults of the irrigation manager. Hence it happens that with the beginning of the operation and maintenance of an irrigation system the most vexing of problems arise, and at such times there is the greatest need of patience and tact, joined with experience and skill.

The opportunities and limitations of irrigation management may be classified under two principal heads—first, the physical, which grow out of the geographic position, character of soil, climate, topography, and water supply; and, second, the human, or psychical, those of the relation of the irrigators to one another and to the management, and of their experience as farmers. The physical conditions are fixed and are susceptible of definite study and fairly complete mastery; the human are variable and by far the most difficult, being almost unfathomable in their complexity.

HUMAN CONDITIONS

Irrigation enterprises, as a rule, have been considered mainly from the physical or engineering side. The promoters or men concerned with irrigation development have approached the subject from the standpoint of the details of building the works. They have had in mind almost exclusively those conditions which pertain to questions of cost, water supply, legal complications, and related factors. It has been a matter of surprise to them to discover, after the works are built and are in condition for operation, that the real elements of success are those more dependent upon proper relations with the farmers and with the soil than they are upon the works themselves. It is, of course, appreciated that for success the works must be well planned and executed; but even when this is done the owners of the irrigation system find that instead of having passed over the most difficult stage of progress they are just entering upon it.

Closely joined with the purely human or sociological elements and relationships with the farmers are the intricate questions of agriculture, the relations of plant growth to water supply, temperature, and other limiting conditions.

Most of the physical conditions are practically fixed; that is to say, the works themselves when built cannot be moved or modified except to a limited extent. Whatever advantages or disadvantages they then possess are beyond radical modification by the management. But the biological or agricultural side, the production of plants and farm animals, the relations with the farmers, their degrees of skill—all of these are constantly fluctuating, are capable of wide modification and come within the range of the activities of the manager and his assistants, either to influence directly or, more often, to modify indirectly.

THE MANAGER'S INFLUENCE

This indirect influence which the manager can bring on the irrigators and their success as farmers is one of the most far-reaching and at the same time most delicate of problems. If the manager is a capable, energetic man, eager to see things move forward, he is constantly tempted to take a strong hand, with the result that he soon finds that he is practically doing the work alone. He is thinking for the community, suggesting and advising, and may suddenly awake to the fact that the energies of other individuals who might have been taking the lead in these matters are expended largely in criticisms of him and of his results. In short, he may discover that his very desire to be of assistance has resulted in demoralizing the community and depriving it of the incentive to self-help.

On the other hand, seeing clearly what should be done, if the manager has a sufficiently diplomatic turn of mind he may be able to stimulate other men to take up these ideas as their own, to push things forward with the enthusiasm which comes from belief in one's own authorship of an idea. It requires considerable self-restraint for an energetic and capable manager to keep in the background and to see other men carry out in part the desirable conditions, but in the outcome it is far more important for the community that this be done than to assume or accept leadership in the activities which do not directly pertain to the management of the irrigation system.

MANAGEMENT AS A PROFESSION

The manager of a large irrigation system has as great a variety of problems as the manager of any large commercial enterprise, such as that of a railroad or of a manufacturing establishment. He must have dealings with large numbers of employees and with his customers and the public in general. There are required for success not only certain qualities of mind such as are included under the terms "common sense and tact" but also years of experience in the construction, and especially in the operation and maintenance, of irrigation works.

The customers or public with which the irrigation manager has to deal is made up largely of farmers, men who as a rule have had little training in business affairs and who do not appreciate the necessity of a thorough and impartial system and of painstaking accuracy in detail. They are apt to resent the requirements which in the abstract they admit are necessary for the success of the community and which should be enforced on others, but which when applied to their individual case put them to some personal inconvenience. For example, if the manager does not enforce the rules regarding wasting of water to prevent the converting of roads into sloughs, the community as a whole has a justifiable criticism, while, on the other hand, the individual farmer who is forced to care for his waste water in the proper manner is righteously indignant at the arbitrary requirements.

There is thus involved in the management of an irrigation system the need not only of skill and experience, but also of firmness combined with tact and rare patience to withstand the continual complaints which reach the manager from aggrieved individuals at all hours of the day and frequently throughout the night. He rarely receives commendation, but rather the contrary, if he is zealous in protecting the interest of the community as a whole. It is this phase of the work which renders the work peculiarly difficult and distasteful to many men otherwise well fitted for it. Nevertheless, in spite of these disadvantages, there is a certain satisfaction in overcoming the difficulties and in exercising the virtues of self-control which in time produce large results.

RIGHTS OF SETTLER

The manager must be constantly alert to see that he himself and his assistants are vigilant at all times not only to observe and study the causes which may lead to discontent, ill feeling or distrust on the part of the water users, but also to see to it that at all times full justice is done to the individual water user. In case of doubt the benefit should be resolved in favor of the man who is trying to cultivate the land. The rights and privileges of the actual settler and cultivator of the soil should be not merely respected, but guarded in his interest, and if any preference is to be shown, it is to the man who is living upon his land and endeavoring to produce crops.

The manager and his assistants occupy a position of double trust in the exercise of their duties—first to operate the irrigation system impartially and in accordance with the provisions of law, for the equal benefit of all irrigators within the irrigation district, and second, for the protection of the interests of the Government, which advanced the funds with which the system was constructed. These conditions should be borne in mind constantly.

Irrigators are at all times to be treated with courtesy and respect, but every employee must remember that the organization of which he is a part serves not only the families in the immediate vicinity in which he is employed, but the whole public. Requests from landowners or other persons to irrigation service employees to act in violation of their instructions will be courteously but firmly refused, the persons making such request being referred to the officer whose duty it is to decide the matter involved.

Irrigation service employees should always be willing to receive complaints, and should forward them for consideration to the manager or chief, in order that if an injustice is being done it may be stopped. In cases where an employee knows of a violation of duty by another employee or of a violation of the provisions of law or of the rules of the service by landowners or their employees, he should report these facts. An employee's first duty is to the irrigation service, and he cannot loyally perform that duty if he allows personal interests or friendships to interfere with it in any conceivable way.

There is a demand for properly qualified men in this profession of management of irrigation systems, principally for assistants or subordinates who will enter at the bottom and gradually work their way up as the opportunity arises for them to demonstrate their ability. In the past most of the principal irrigation managers and their assistants have entered the profession from the engineering side, being originally connected with the construction of the works. They have gradually acquired experience in handling the works as they approach completion and are put into use. A large irrigation system is so complicated that at first there must be a number of men connected with the management who are thoroughly familiar with the construction of the different parts and who have had the opportunity of watching the testing out of these during the early stages. In the hundreds of miles of canals and laterals and scores of structures there is so much of the work which is out of sight, covered up by earth, and so many points of structural weakness or possible danger that the men who designed and built these portions can alone be trusted to get them into active operation. One of the greatest dangers to the new piece of work is the attempt of new men unaware of many of the conditions to handle the work.

WHO MAKES BEST EXECUTIVE?

In the choice of irrigation managers it is not necessary or desirable to confine the selection wholly to the engineering side. In fact, there is a need for men who have been brought up on a farm and who have had the practical experience of an irrigation farmer. Especially valuable are the young men who, in addition to their life on an irrigation farm, have had a good training in the theory of agriculture at a properly equipped college. Such men, combining a little engineering experience with an agricultural course, should be the ideal assistants in the management of an irrigation system.

There is a third class of men from whom good managers have been drawn—namely, the clerical class, or the men who have come into the work as bookkeepers or accountants, who have had some experience as inspectors on the ground and who have acquired a business training and experience which fits them to handle the larger problems of management, especially if provided with experienced engineering assistants.

There are thus three lines through which men enter the profession of irrigation management—first, through the engineering; second, through the agricultural; and, third, through the clerical. The final success is determined more largely by the personal qualities of good judgment and tact than by the limitations of early experience along any one of these lines.

The colleges devoted to instruction in agricultural and mechanical arts have given more or less attention to the elements of hydraulic engineering. Those in the Western arid States have, in outline at least, a course in irrigation engineering. The larger Eastern technical schools have also touched upon irrigation work in connection with various branches of civil engineering. There has not been, however, any general recognition of irrigation management as a distinct profession for which men are to be trained by a systematic course of observation and study, but, as before stated, the irrigation managers have been the product of evolution, largely from the constructing force.

The main problems of management are, however, so distinct from those of engineering construction that there is need of a recognition by the agricultural schools of a training for such men which is primarily agricultural or biological in character. The engineering problems of management, while in themselves complicated, are not as far reaching as those having to do with the human or agricultural side of the work. They may be more safely intrusted to assistants or to occasional advice from experienced constructing engineers.

The ideal course of instruction would be one given to young men who have been brought up on the farm. This should include the theory and practice of agriculture, with the elements of hydraulic engineering, and with instruction in a few of the fundamental conceptions of water laws.

For lack of more complete training a correspondence course in well-established colleges is of considerable value and assistance to the younger men employed as canal riders or water masters and who are ambitious to perfect themselves along these lines and lay the foundation for future advancement. Such courses are now being offered in several of the Western State colleges and are being followed with success by enterprising young men who could not otherwise enjoy the advantages of instruction along these lines.

EFFICIENCY AND ECONOMY

The motto of the manager of an irrigation system is: "Efficiency first, then economy." Good service must be rendered; when this is done, then every possible effort should be made to maintain it in the most economical manner.

The development of an efficient system of management must come about through careful study of all surrounding conditions and experience such as can be gained only as the works are actually operated. The details of effective service cannot be determined in advance but must be worked out upon the ground in conformity with the requirements of each case, and of adjustment of these to laws and local regulations.

It is highly important in these matters to secure the co-operation of the water users themselves. To do this it is necessary, first, to work out as nearly as possible what seems to be the best system and then, having clearly in mind the requirements of the case, consult with representative water users. In other words, the management must first know what can be done, then make such adjustments as may be practicable to suit the ideas of the water users themselves, whenever it is possible to make such adjustments within the limit of effective operation.

Overirrigation and underfertilization, with resulting injury to the land and waste of water and efforts, are the fundamental difficulties to be overcome in securing efficiency and economy. With skill and patience the irrigation manager gradually achieves results, educating the water users to an appreciation of these evils and slowly securing their co-operation.

From the foregoing general review of irrigation management as a whole it is seen that there is an opportunity for an engineer to display not merely professional skill, but to utilize all of his abilities as a business man and a good citizen. Assuming that he does make progress, the manager has the satisfaction of genuine achievement, of having done a man's work.

Hints on How to Develop Export Trade with South America

Interviews with E. N. Chilson on Credit Relations and with E. F. Colyer on Trade-Getting Methods—List of Publications Containing Information of Help to the American Exporter

Better Credit Relations Must Precede South American Trade Increase

E. N. Chilson Warns American Manufacturers Against Raising Prices to Secure Temporary Profits, Thereby Destroying Confidence

STATISTICS cited by E. N. Chilson, purchasing agent of J. G. White & Company, of New York, in an interview with the Engineering Record's representative, show that 75 per cent of South American imports in transportation materials, power-house apparatus, coffee machinery and agricultural machinery is either manufactured by Europe or sold by Europe. The countries mostly concerned in this are Argentina, Brazil, Chile and Venezuela. The credits of the South American consumers or dealers have been so long established in Europe that it is only natural that they should seek goods there. The usual credit given is ninety days' draft through European markets. This gives the purchaser an opportunity to place an order and have it delivered to him for consumption before actual payment. On the other hand, the American method provides for three days' sight draft, thereby making the purchaser assume most of the risks. If we are to establish American trade with South America on a firm basis, however, we must bring about an amicable credit adjustment so that South Americans can come direct to our markets, get goods shipped and start actual disposition of goods before payment is made.

DIFFERENCE IN BUSINESS METHODS

A Brazilian paper manufacturer recently endeavored to buy 10,000 tons of paper through New York brokers. He had formerly paid German exporters on a ninety-day draft, but New York brokers refused to sell except on a confirmed bankers' credit or a gold basis. If the American paper company had had an agent in Rio de Janeiro to look up this customer's credit it would have been possible to make a satisfactory adjustment.

It has been suggested that a sufficient trade balance be established in New York to allow the South Americans to purchase all goods needed. Were this condition once established, Europe would not be able to take trade back after the war. It is the prevailing opinion here that if the war continues from twelve to eighteen months industrial and agricultural pursuits in Germany, England, France and Russia will be so demoralized that it will take an equal or longer time for these countries to recuperate. Hence, American manufacturers will probably have a period of about three years in which to place themselves on a firm basis with South American consumers. The countries engaged in South American trade were ranked before the war in regard to volume of trade as follows: (1) England, (2) Germany, (3) France, (4) Belgium, and (5) United States. If American manufacturers make the proper use of their opportunities America should take a much higher place in the list.

TRANSPORTATION FACILITIES

In regard to proper transportation facilities, it may be said that the Brazilian Government has a Government-owned steamship line, the Brazilian Lloyd, of 100 vessels, and with the trade balances coming our way, the line will no doubt be placed in the American trade. Already Ambassador Morgan has cabled from Rio de Janeiro that the Minister of Finance of Brazil is arranging to dispatch to the United States five vessels of this line, with consignments of coffee from Santos and Rio de Janeiro and rubber from Para, which are to bring back cargoes of prime necessities.

American manufacturers, however, must take a long-sighted view of the present situation and not endeavor to make a temporary profit by boosting prices. South American manufacturers would feel that they were being fleeced on account of the almost compulsory trading with this country, and after the war they would turn back instantly to their former dealers. The rail market, for instance, has been dead for the last two or three years. Orders from South American trade could be easily handled by American steel manufacturers and would serve to make up for the present depression in American trade. South American consumers are well aware of this fact, and any attempt to run up the price through a falsification of the facts of present demand and supply would only result in irretrievably losing the South American market when the present war restrictions on European products are removed.

Intimate Details of South American Trade-Getting

E. F. Colyer Gives the Results of His Experience on Such Points as Establishment of Relations, Traveling Expenses and Time for Trips

E. F. COLYER, in charge of the Latin-American section of the foreign department of the General Electric Company, has had much experience in Central America and South America. In discussing export trade matters with a representative of the Engineering Record he said that no commercial traveler who went to either Central America or South America could do much if he did not know the language spoken in the country he visited. In all countries excepting Brazil, where Portuguese is spoken, and in British, French and Dutch Guiana it is necessary to talk Spanish. Although some of the plants are owned by the English or Americans, the miscellaneous trade, except in the large cities, is principally in the hands of native firms, and with these a foreigner who had to talk through an interpreter could make very little progress, Mr. Colyer believes. This advice, of course, applies to travelers who are trying to make actual sales. If a representative of an American manufacturer goes to the country merely in order to get information concerning the outlook for his goods it is not so imperative that he speak the language. It will be found that many of the plants and stores in Central American countries have connections with commission houses in New York and New Orleans.

In offering goods to residents of the Central and South American countries it is most desirable, Mr. Colyer stated, to have letters of introduction in order that a friendly relation may be established before actual business is attempted. As the banks of this country do not often have correspondents there, they are not likely to be of much assistance to commercial travelers. If it is possible to get a letter of introduction to a resident banker or to establish a business connection with him that is an advantage. In all cases the first man to be seen is the consul. American consuls are always glad to assist any man who is on a business trip and frequently they are able to give much information that is of service. A representative of a large well known corporation is in a somewhat different position from one that represents an unknown concern. Correspondence between a company and a possible consumer is, of course, a step toward a personal introduction that will be found to be a help.

TRAVELING EXPENSES

Mr. Colyer emphasized the fact that much of the business with the greater part of Central and South America is apt to be small in amount and that some time is required to develop the market. He figures the traveling expense of a salesman in Central or South American countries as pretty close to $5,000 a year. If a salesman were to attempt to cover the entire field he would find the territory so large that he might easily become discouraged. Business is not transacted in accordance with the practices that prevail here. There is little prospect that several orders a day may be taken. Often a salesman will not receive a single order until he has been on the ground for weeks.

Mr. Colyer said that with the possible exception of Guatemala, which is more nearly a market for European countries, the Central American countries patronize mainly the manufacturers of this country. Between the commission and jobbing houses and local representatives of American firms, Central America is now pretty well covered by manufacturers of this country.

Mr. Colyer was asked about the time of the year when trips should be made by representatives of manufacturers. He said that it varied with the section visited, but that it was desirable to avoid the rainy season. Clothes suitable for the tropics should be taken, with a light overcoat for the altitudes.

The question of granting credit, Mr. Colyer believes, is not radically different in Central and South America from other countries. A good deal of misinformation has been published about the general necessity of very long credits, which is not the invariable rule. In some lines extended credits must be given, while in other lines it would be poor business policy. The practice in different markets and in different lines varies very widely.

Since the foreign trade of Brazil, the Argentine, and Chile is several times as great as that of the rest of Latin America, these countries naturally offer the most attractive market for American manufacturers. They have in the past been supplied principally from Europe. They have large modern cities, are well provided with railways, and business methods will be found to differ less widely from ours than is the case in other sections.

The question of the form of package and method of packing has been discussed often in connection with Central and South American trade. Mr. Colyer said that the important point in packing for export is to make the package so carefully that goods will not be broken. In some countries buyers stipulate packing for muleback transportation. If this is mentioned it is absolutely imperative that the instructions be followed. On the west coast the conditions necessary to meet in landing freight are very bad and stout packing is required. In other countries, where duties are assessed in accordance with the weight, buyers stipulate that the package be as light as possible.

Publications on South American Trade

Bulletins Available from Bureau of Foreign and Domestic Commerce and National Association of Manufacturers

MANUFACTURERS and others seeking to take advantage of the present favorable opportunity to introduce their products to the South American markets will find certain publications of the Bureau of Foreign and Domestic Commerce, Washington, of assistance. The bulletins deal with special features of the commerce of South America and include series of consular reports on particular subjects, reports of investigations by special agents of the Department of Commerce, etc. A full list of these with a synopsis of each has been printed in pamphlet form and will be sent free on request to the Bureau. Attention is also directed to the consular reports from South America appearing constantly in the "Daily Consular and Trade Reports." The following list gives the publications referred to, most of which can be obtained for a small sum from the Superintendent of Documents, Government Printing Office, Washington, D. C.:

Commercial Agents' Reports:

Trade Conditions in Brazil, by Lincoln Hutchinson; 1906.

Trade Conditions in Argentina, Paraguay, and Uruguay, by Lincoln Hutchinson; 1906.

Trade Conditions in Central America and on the West Coast of South America, by Lincoln Hutchinson; 1906.

Trade Conditions in Colombia, by Charles M. Pepper; 1907.

Trade Conditions in Ecuador, by Charles M. Pepper; 1908.

Trade Conditions on the West Coast of South America, by Charles M. Pepper; 1908.

Trade Development in Argentina, by James Davenport Whelpley; 1911.

Trade Development in Latin America, by John M. Turner; 1911.

Foreign Credits, by Archibald J. Wolfe; 1913.

Transportation Rates to the West Coast of South America, by F. J. Sheridan; 1913.

Special Consular Reports:

Winning Foreign Markets; 1908.

Coal Trade in Latin America; 1910.

Foreign Markets for Railway Supplies and Equipment; 1913.

Miscellaneous Publications:

Packing for Export; 1911.

Factors in Foreign Trade; 1912.

Foreign Publications for Advertising American Goods; 1913.

Trade Directory of South America; 1914.

Tariff Publications—The Division of Foreign Tariffs of the Bureau of Foreign and Domestic Commerce has issued a series of publications dealing with various tariff schedules, consular regulations, taxes on commercial travelers, etc. The Bureau of Foreign and Domestic Commerce will also furnish information on request concerning the rates of duty on any articles in South American countries.

The foreign trade department of the National Association of Manufacturers, 30 Church Street, New York, has issued a "Special Trade Bulletin," dated August 17, intended to keep members promptly advised of all developments affecting the export and import trade arising out of the war. Further issues of this publication will be made as occasion warrants. The contents of Bulletin 1 are: The war's effect on American trade; banking credit and exchange; present shipping conditions; marine insurance rates; contraband and neutral trade; loss of goods in transit; moratorium defined; foreign collections and proposed new shipping laws.

Packing Goods for Foreign Shipment

PROPER packing, it has been frequently emphasized, is essential if foreign customers are to be held. In this apparently small, but really important detail, European manufacturers have excelled their American rivals. These views were strongly emphasized by W. E. Pearson, of the Pearson Engineering Corporation, New York, in an interview with a representative of the Engineering Record. He remarked that the importance of packing was due to the number of transshipments necessary before goods reach their destination. Enlarging on this topic Mr. Pearson said:

"The large American manufacturers of electrical apparatus, like the General Electric, Westinghouse and Western Electric, understand the necessity of this, and their shipments rarely are received in bad condition. It would seem as if they had learned how to pack lamps, for instance, by trying different ways and then throwing the packages out of the window until one kind of packing has been devised which would stand such treatment. But most American manufacturers have not yet learned how to pack their goods for South American delivery. Thus, in a shipment of glass which we received some time ago from an American manufacturer, 99 per cent of the glass was broken. We could get no redress from either the shipper or the shipping company. I know of only one manufacturer of gas stoves that can pack these goods so that they will be delivered in Brazil in good condition."

Amended Brick-Paving Specifications

Addition of Rattler-Test Requirement and Clause Requiring Lugs, and Prohibition of Any but Batch Mixer, Feature New Rules

CARRYING several clauses dictating practice varying from that suggested in the previous edition, revised specifications for building brick street pavements and highways have been issued by the National Paving Brick Manufacturers' Association, of which Will P. Blair is secretary. The principal changes include the addition of the rattler-test requirement, as well as of a clause providing for lugs on all brick; specification of the batch mixer only for all concrete work, and the elimination of many dimensions and proportions hitherto plainly stated.

The main clause referring to the rattler test reads as follows: "The bricks shall not lose of their weight more than ...... per cent, after being submitted to the Standard Rattler Test, to be made under the specifications in the manner and method and with a rattler together with the records thereof, as proposed by Committee C-3 of the American Society for Testing Materials at their annual meeting in 1913, as herein set forth. The permissible loss to be specified in any given district or municipality and for any given purpose is a matter wholly within the province of the buyer, and should be governed by the kind and amount of traffic to be carried, and the quality of paving bricks available." Qualification and refinement of this injunction are given in other clauses, which explain at length equitable procedure for accepting or rejecting the blocks offered.

SPECIFICATION OF LUGS

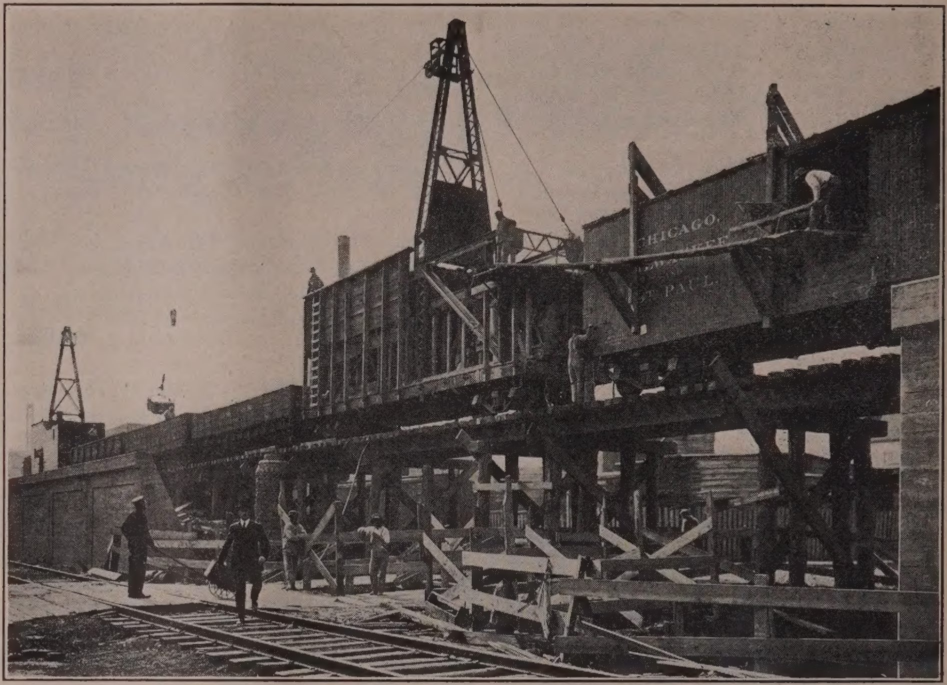
Lugs are specified in the following sentence, taken from a section referring to quality: "They (the brick) shall have one fairly straight face and be provided with not less than two or more than four projections on one side of the brick, which shall not project more than one-fourth of an inch nor less than one-eighth of an inch."

In former editions of the specifications standard curbstone sizes, the depth of foundation, and the proportions of the mixture of the foundation or of the curbing, if of concrete, were plainly stated. In the new issue, however, reference to some of these has been entirely omitted and blanks have been left where others should occur, evidently in recognition that the growing understanding of brick paving essentials allows determination of these figures to conform to local conditions.

Many minor changes are apparent in the newly-drafted rules. It is now preferred that the courses be laid at right angles to the center line, whereas formerly either a 90-deg. or 45-deg. angle was allowed. Attention is called to the importance of thorough drainage by two sketches of cross-sections as well as by a rule requiring quick removal of water to a depth of 18 inches; and the maximum weight of the roller to be used for compacting the sub-base has been increased from 8 to 10 tons, the drafting committee apparently no longer fearing that the larger machines are too cumbersome. A prohibition against retempering mortar or concrete has been included and the process of bringing the 1½-in. sand cushion to proper cross-section with a template has been treated with care.

Where cement is mentioned it is stated that the requirements of the American Society for Testing Materials are to be met. In following the lead of the last National Conference on Concrete Road Building in declaring for the batch mixer, the brick manufacturers have adhered to the conservative requirement that cement and aggregates should be mixed dry. An innovation has been made, however, in naming the cement unit to be used in concrete mixtures. Instead of stating that......cubic feet of cement were to be used, the stone having always twice the volume of the sand, the requirement reads that "the resulting concrete shall contain fine aggregate amounting to one-half of the volume of the coarse aggregate, and that......cu. ft. of the concrete in place will contain 94 lbs. of cement." In view of the standard character of most brands of cement the advisability of this refinement may be questioned.

The concrete foundation has been definitely given the place of honor in the new rules. Formerly attention was directed to a foundation of paving block, which was plainly stated to have been proved the stronger base. No traffic should be allowed on the street, state the specifications, for twelve days after the ½-in. layer of sand has been added to the completed surface—an increase of two days over former requirements. It is also directed that the contractor must insert thin metal strips 1/16 in. x 6 in. x 3 ft. in the brick joints across the street when closing a stretch of grouting at work intervals.


DELIVERING BULK CEMENT AND AGGREGATES FROM CARS TO CONCRETE MIXER

Placing 300,000 Cubic Yards of Concrete

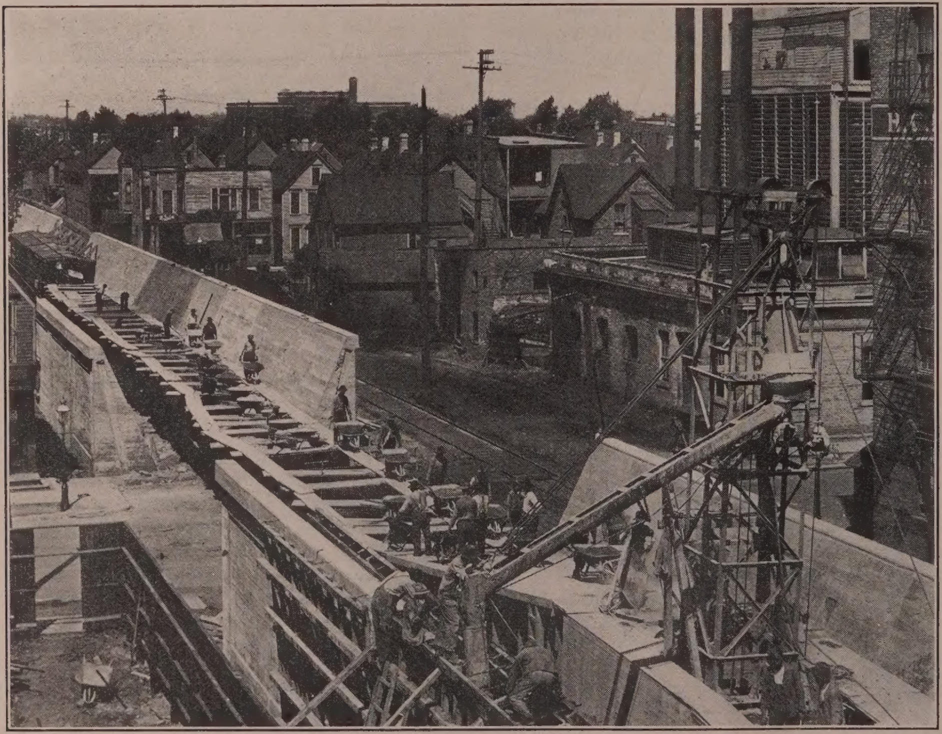
Special Plants and Extensive Use of Bulk Cement on Track Elevation of Chicago, Milwaukee & St. Paul Railway at Chicago

RETAINING walls 4.8 mi. long on one division and 8½ mi. on another, together with seventy-one subways, involving more than 300,000 cu. yd. of concrete now being put in by seven construction trains and more than 1000 men at the rate of 1000 cu. yd. per day, are among the features of the track elevation of the Chicago, Milwaukee & St. Paul Railway on the Council Bluffs division and the Evanston division in Chicago, in progress since last year and to be continued until 1917.

Complying with the ordinance requiring the elevation of the 2.4 mi. of double track in Bloomingdale Road on the Council Bluffs division by 1915, $2,000,000 was appropriated and $3,000,000 more will be required in connection with the 4¼ mi. of construction on the Evanston division, due to be completed in 1917; in addition to which there will be further work on the Chicago and Milwaukee division. One-half of the 135,000 cu. yd. of concrete estimated for the Council Bluffs division have been placed to date by means of the five construction trains employed on Bloomingdale Road. Special equipment on this work was described in the Engineering Record of June 21, 1913, page 701, and Nov. 15, 1913, page 544. A fourth of the 200,000 cu. yd. to go into the Evanston division work are to be poured this year by available trains, of which two are now on the job.

The railway company has made a point of employing up-to-date construction methods, including specially built apparatus and the use of material to the best advantage. The satisfactory results and economy demonstrated in the use of bulk cement shipped from the nearby Buffington mills in 1913 resulted in its being the preferred material in the 1914 specifications. This is believed to be the largest use to date of bulk cement.

There are four essentially dissimilar types of concreting plants in operation on the work, varying from the ordinary mixing train chuting concrete directly to the foundations and the locomotive crane for elevating to the tops of wall forms to the most recent developments of tower elevator and portable dragline. For the foundations it was found that no piling would be necessary, since the clay subsoil had a safe load capacity of 4000 lb. per square foot. The decision to build the retaining walls from ground level instead of driving trestles in advance involved lifting the concrete by suitable elevator, and for this the usual movable cranes have been supple-


POURING CONCRETE RETAINING WALL IN CLOSE QUARTERS

mented by the knock-down tower elevator with swinging spout furnished by W. B. Louer, of Chicago, and described a year ago in the earlier of the two articles mentioned.

In transferring materials to the mixer the general practice of wheeling the aggregates on runways over the gondolas and of similarly wheeling bulk cement from the box car, located preferably adjacent to the mixer, has been applied on all the construction trains except one, and in this a radical departure from previous practice has been instituted by operating a dragline over a train load of material between two portable towers, as described in the issue of Nov. 15, 1913. The device has been materially improved since that description was written, shortly after its initial use.

RETAINING WALLS

Most of the retaining walls are to be 17 ft. high, erected in 25-ft. sections, of thickness graduated from 1½ ft. at the top to 8 or 10 ft. at the base, and supported on footings from 2½ to 3¾ ft. deep and 11 to 13 ft. wide. On Bloomingdale Road the comparatively narrow right-of-way, 32 ft. wide, necessitates full retaining walls on both sides. Of the thirty-five subways to be constructed on this division, all are of reinforced concrete, erected in short spans with flat slabs, with the exception of three exceptionally wide intersections, where structural steel will be employed. On the Evanston division there are to be thirty-six subways and the 60-ft. right-of-way will carry either three or four tracks, dependent on the possible traffic of the electric roads to be accommodated on this structure. The Northwestern Elevated Railway will require seven passenger stations in handling the passenger traffic.

The construction trains engaged in chuting concrete to footings are progressing at the rate of at least 100 lin. ft. per day and employ crews of about thirty men. The ½-yd mixers in this connection are observed to turn out a batch of concrete at intervals averaging 1 min., so that while they have no difficulty in exceeding the minimum standard of 150 cu. yd. per day they are evidently capable of producing 50 per cent more than is required of them in a full 10-hr. day, actual mixing time.

For wall pouring, the locomotive-crane type of elevator is observed to place ½ yd. at intervals of approximately 1½ min. In this connection the crew is practically the same as in the preceding instance, with the addition of the steam engineer and fireman on the hoist.

There are seven concrete mixers in use on the work—two Chain Belts, two Smiths, a Ransome and two Chicagos.

With the use of bulk cement one man is eliminated from the charging platform, and when possible to do so, locating the cement car immediately adjoining the mixer permits a further reduction of two men in the number of cement wheelers as compared with those required when the cement car is at the extreme end of the construction train, as is shown from observations made. The wheeler's time in transit from car to mixer was 15 sec. from the adjacent car, 30 sec. from the second car, 40 sec. from the fourth car, and 1 min. 15 sec. from the seventh car, at the train end. A comparison of the round-trip time, 2 min., of a wheeler when the cement car is next to the mixer with that of 4 min. when the car is at the opposite extreme indicates the possible reduction of two men, which would mean a difference of $4 per day, or 2½ cents per yard, in the production of 150 cu. yd. per day by one machine.

The efficiency of the tower elevating plant is such that the unit cost of concrete is from 7 to 10 cents a yard less than in the preceding instances. The machine made a record of 200 cu. yd. per day in May, 1914. There is practically no extra labor cost in connection with this plant, the mechanical equipment of which includes a 25-hp horizontal boiler, a 20-hp hoisting engine, and ½-yd. mixer, besides the steel tower which contains the elevating bucket and rises 30 ft. above the car body so as to support the movable steel chute.

The economy of operation of the dragline is stated to be about one-third better than that of any other plant, since the mixer has been mounted on the end of the hopper car and the aggregate is conveyed to the charging hopper of the mixer by means of a belt conveyor placed underneath the 30-cu. yd. storage bins. The crew numbers not more than fourteen, including the sand loaders, necessary only part of the time, and two cement wheelers and a loader, who transfer bulk cement from an adjacent box-car to the mixer. It is found that 75 per cent of the material in a gondola may be removed by the drag bucket without assistance and 60 cu. yd. may be handled in an hour. This train is now pouring concrete in the subways by chute. Working at full efficiency at the rate of 25 cu. yd. per hour it will place 250 cu. yd. per day.

In first cost the dragline, at $6,000, is more expensive than the $3,000 tower elevating machine, but less expensive than the $8,000 locomotive crane. The productive capacity and economy of operation seem to give preference to the first named. In this connection it has been pointed out by those in charge of the work that by the use of bulk cement there is possible a reduction of any crew by two men, which would be a saving of nearly 3 cents per cubic yard on a 150-yd. output per day, while the elimination of sacks means a reduction per barrel of 2¼ cents in price, 2 cents previously chargeable to sack loss and 1 cent previously chargeable to bundling and freight handling.

This would mean a total difference of between 5 and 10 cents per barrel, which on the prospective Evanston construction alone would amount to probably $15,000, while a proportional sum may be saved on the work of the other divisions.

The work is under the direction of the company's engineering force, of which C. F. Loweth is chief engineer; R. J. Middleton, engineer of track elevation; H. C. Lothholz, engineer of design; C. H. Buford and F. C. Loweth, assistant engineers, Bloomingdale Road elevation, and J. E. Weston, assistant engineer, Evanston division.

DYNAMITE inserted in lengths of condemned fire hose was used to cut to pieces the steel drill barge "Teredo" which had been sunk at Panama by an accidental explosion. It having been found impracticable to raise it, it was decided to cut it to pieces. The hose with its charge of explosives was placed along the line at which it was desired to direct the force of the blast and, according to the "Canal Record," made a fairly clean cut. The sections sheared off were raised by wrecking barges assisted by a dipper dredge.

London Asphalt Pavements Expensive

Averages for Rock Asphalt and Wood Block in Westminster and London Proper Show Latter to Be 15 Per Cent Cheaper

RELATIVE costs for wood-block and asphalt pavements in two divisions of Greater London, as given in the report of George W. Tillson, consulting engineer to the president of the Borough of Brooklyn, New York City, will surprise some American readers. The conclusions of Mr. Tillson's report were summarized on page 227 of the Engineering Record of Aug. 22.

The street system of London proper, which covers about 1 sq. mi., he states, contains 48.5 mi. of streets, 38.75 mi. of which are paved—8.5 mi. with wood blocks, 20 mi. with asphalt, and 10.25 mi. of granite blocks. It is said that the wood block does not last as long as the asphalt, but costs less for repairs. Asphalt, on a 9-in. concrete base, costs $3.81 per square yard, and the wood $3.20 per square yard. The average repair cost per year for asphalt is 30 cents per square yard, and for wood 20 cents, although some streets cost as high as 43 cents. A contract was let for keeping the asphalt pavement in Cheapside in repair for fifteen years after it had been laid two years at 56 cents per square yard per year.

TYPICAL WOOD PAVEMENT

The repaving of Grace Church Street with wood was inspected. Here an old asphalt pavement, with a 9-in. foundation, had been entirely taken up and replaced with wood-block pavement on a 9-in. foundation. This work was being done at a cost of $3.25 per square yard. The roadway was 34 ft. wide and two courses of blocks were laid parallel with the curb, with an expansion joint of clay between them. The concrete was finished extremely smooth and the blocks were laid directly upon it, one man laying 200 sq. yd. per day. The joints were then pitched and covered with cement grout, and received a final covering of gravel. The engineer in charge stated that this pavement was expected to last five years.

In Westminster, which has 99 mi. of paved streets, asphalt pavement laid on a 9-in. concrete foundation cost in 1913 $3.46 per square yard. One street was surfaced in 1912 with a 2-in. layer, at a cost of $1.93 per square yard. The price of the wood pavement in 1912, including foundation, ranged practically from $2.10 to $3.15 per square yard. The price of $2.10, however, was for one street only, the next higher being $2.65, and the average a little over $3. All of these pavements were laid with a 12-month guarantee only, 10 per cent of the contract price being retained for that length of time. The cost of asphalt repairs runs from 12½ cents to 62½ cents per square yard, and of wood-block repairs from 12½ to 37½ cents per square yard.

The heavy 9-in. foundation was adopted in Westminster in 1913. Rock asphalt is used for all these streets, the specification in Westminster reading as follows:

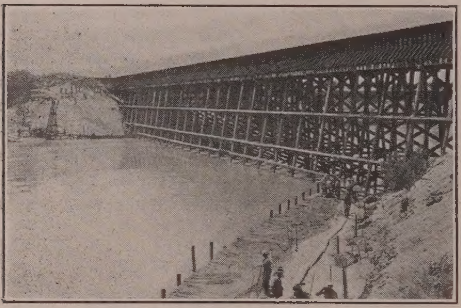
"Rock asphalte shall be a natural bituminous limestone rock of the best quality, without the addition of any other substance whatever, ground to a fine powder, heated to the proper degree, and thoroughly compressed with hot rammers, or rolled to the uniform thickness specified."

Riprapping with Weed Bundles

Embankments in Imperial Valley Protected by Arrow Weed Bound in Sheaves and Placed with Butt Ends in Water

IN maintaining the levees of the Imperial Valley irrigation system and protecting channel embankments the chief problem is most often that of securing a facing which can be built quickly and cheaply and which will resist the erosion of the current. Stone is not often to be had as cheaply or as quickly as desired, and the best soil available is so friable that bags of it placed as a facing are likely to be robbed of their filling by the action of the current.

In the absence of better material the expedient of using bundles of arrow weed to form the facing of the earth embankments is said to be very successful. Large quantities of the weed can usually be found in 5 or 6-ft. lengths, and this is cut and made into bundles about 15 in. in diameter. The bundles are secured by a single wrap of baling wire at about the midpoint, and are then placed in horizontal layers with the butt ends to the water. In some cases bags of earth are piled on the bundles for about 4 ft. back from the outer ends. Behind the bags loose earth is thrown on the weed tops, building up the embankment as fast as the facing is raised.



RIPRAPPING WITH BUNDLES

Where bags of earth are not used the weed bundles are usually held in line by wire strung between small piles driven along the embankment. Piles of some sort are nearly always used along the outer edge of the facing, and vary from light willow poles to 6 x 6-in. timbers spaced 5 ft. apart. Although no definite records are available, engineers report having taken out weed riprapping which had served effectively for long periods. The weed does not decay quickly, but the bundles pack down until the embankment is almost a solid mass.

The accompanying illustration shows an arrow-weed embankment about 200 ft. long being thrown up to protect Encina flume when threatened by the Volcano Lake levee break of June 11 (described in the Engineering Record of June 20 and July 4). In this case the riprapping was started before the water had advanced this far, and was carried up as required, the total height being about 6 ft. For every three of the piles shown, which are 4 x 4 in. in section, there is a short 6 x 6-in. post driven further back in the embankment as an anchor and tied to all three piles by wires.

The flood water reached this point on June 19, and the maximum flow, which occurred on June 21, was calculated to be 6000 sec.-ft. The arrow-weed embankment is reported to have been in good condition after the flood subsided.

Atomizing Paint on Rough Timber

1000 Square Yards Painted Per Man Per Day in Palaces of Panama-Pacific Exposition, San Francisco

SIX of the main palaces of the Panama-Pacific International Exposition require to have the interiors painted, while the others are to be left in the rough-timber finish. A total interior surface of approximately 700,000 sq. yd. is to be painted, and because of the roughness of the unfinished lumber in many parts, as well as the inaccessibility of the surfaces to be covered, the ordinary methods would not have sufficed to complete the work within the permissible cost and time.

Work is now in progress, however, in the Palace of Agriculture on a contract which allows only 100 days for painting the entire interior of the six structures. The rate of progress thus far is reported to be 800 to 1000 sq. yd. per day per man, or about eight to ten times the normal rate with hand-brush on finished surfaces. The method used is to transmit paint under pressure to all parts of the building through a system of pipes and to apply it to the surface directly from nozzles operated by the painters. The equipment installed in the Palace of Agriculture is capable of supplying twenty-six operators continuously.

Equipment

There are two separate piping systems used in the work, one for compressed air and the other for the paint. A 1¼-in. material or paint main runs down the center of the building supplying laterals at convenient points, while a 2-in. air main paralleling the paint line supplies pressure to an equal number of laterals. Each pair of laterals terminates in a 5-gal. tank suspended near the operator, and from it a pair of flexible lines run to the nozzle, which the operator moves about at will, usually about 18 in. from the surface being treated. The 5-gal. tank acts as a small storage reservoir, and carries valves by means of which the amount of air or material allowed to enter it can be regulated. One of the flexible lines takes air from the tank, while the other has a submerged intake and conveys material only, so that the air and material reach the nozzle through separate lines.

The nozzle is designed to atomize the liquid by the manner in which the air jet takes it up, and the flow of air and material can be regulated independently by the operator by means of a valve in each tube just back of its junction at the nozzle. Thus the liquid in the shape of fine particles resembling mist is thrown forcibly against the surface to be covered, and the density of the application is regulated by the control valves to suit the condition of the surface under treatment.

Materials

The paint is compounded according to a secret formula, but it is said that it contains magnesite as a fireproofing; and as it is mixed by simply adding cold water to a dry compound, the absence of oil as a vehicle is evident. It is estimated that 175 tons of the dry material will be required for the entire contract. The rate of consumption in the Palace of Agriculture has been about 25 bbl. of the dry material per day.

The mixing is done on the main floor in a tank alongside of which is an electrically driven air compressor with a capacity of 190 cu. ft. of free air per minute. This supplies twenty-six operators, each equipped with a 5-gal. storage tank. A pressure of 30 lb. per square inch is maintained continuously in both air and material lines.

The maximum distance which air and material are transmitted is 1500 ft., and the highest point served is in the dome, 176 ft. above floor level. The latter height is reached by the aid of an auxiliary hand pump, which increases the pressure where necessary, the 30-lb. standard having been found most advantageous for the major portion of the work.

The contract for painting the interiors of the six palaces is held by the Progressive Painting Company, of San Francisco, which has developed the process, to be known as "Tilmanizing."

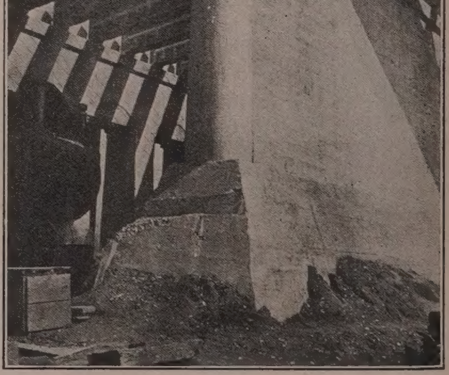
Coon Rapids Plant Put in Operation

THE first 2100-hp unit of the Coon Rapids hydroelectric development was placed in commercial operation Aug. 1. The unit is the first of five, all of which will be completed and placed in service by Nov. 1. Construction of this development was started in February, 1913, by H. M. Byllesby & Company, of Chicago, and the main dam was completed Jan. 7, 1914. The development was described in the Engineering Record of Jan. 17, 1914, page 77.

An Artistic Bridge Pier

Concrete Support for Railway Crossing Treated to Simulate Circular Column

A CONCRETE bridge pier on the Chicago, Rock Island & Pacific Railway has been treated so as to make its end simulate columns. The pier is located at the crossing with the Chicago & Western Indiana Railroad at Seventy-ninth Street in Chicago. The accompanying photograph shows the artistic appearance of the work.



PIER TREATED TO SIMULATE COLUMN

Design of Pole Foundations

Derivation of Formulæ Based Upon the Compressive and Frictional Resistance to Overturning Offered by Ordinary Soil

By C. L. CHRISTENSEN
Cincinnati, Ohio

POLE foundations are mostly designed according to mere guesswork, and therefore not economically or safely. Considering that many thousands are being built every year and will be built in increasing numbers as the construction of power transmission lines progresses, the writer has thought that some suggestions made on a rational basis may be of interest. They apply to ordinary soil, but not to quicksand or swampy ground. They are made with reference to rectangular pole foundations; the method, however, can be modified to foundations of more irregular shape.

Let P signify the resultant of the overturning forces; h, height of P above ground; d, depth of foundation, and M, overturning moment in regard to the bottom of the foundation. Then

$$M = P(h + d)$$

After a tentative design has been made the various moments from the pressures on the sides and bottom of the foundation, which tend to resist the overturning, can be estimated as follows:

Let W = dead load of foundation and superstructure, in pounds; b, width of foundation at right angles to M, in feet; c, width of foundation parallel with M; g_1, ultimate soil-pressure intensity in pounds per square foot at bottom of foundation. The ultimate soil-pressure intensity will most frequently not be reached before the center of pressure on the bottom has been moved outside the middle third of c. This is because the load from the superstructure is small. Assuming this, the distance from center of pressure to front edge of foundation is $2\,W/3\,g_1 b = f$. Let e signify the distance from W to the front edge. Then the ultimate resisting moment from W and its reaction is $W\,(e - f)$.

RESISTING MOMENT FROM PASSIVE EARTH PRESSURE

The resisting moment from the passive earth pressure on front side of foundation is next to be considered. When the foundation tips forward around a point at depth t the deformation curve is a straight line, with no deformation at depth t and maximum deformation at the ground line. On the other hand, the ultimate passive resistance per square foot of earth against deformation horizontally is zero at the ground line and can be assumed to increase in linear proportion to the depth.

The actual pressure intensities are obtained by multiplying the deformation in a certain depth with the corresponding resistance against deformation. The actual pressure intensity is therefore zero at the ground line and zero at depth t. Between these points the pressure intensities vary as the abscissæ of a parabola with axis horizontal at the depth $\frac{1}{2}t$. Let δ signify the ultimate intensity of passive earth pressure at a depth of 1 ft. We will then as ultimate passive earth pressure on the front of the foundation use the value

$$\frac{2t}{3} \times \frac{t\delta}{2} \times b = \frac{t^2\delta b}{3}$$

In doing so we have considered that the layers above $t/2$ are forced to slide a little, but also that the layers between $t/2$ and t have a passive resistance not fully utilized. The pressure intensity at depth $t/2$ which would correspond to no sliding of any layer—not even those nearest the ground line—is $t\delta/4$.

If the point of rotation is located at the bottom of the foundation the passive pressure is distributed over the whole front of the foundation and is $\dfrac{bd^2\delta}{3} = E$. This will be the case if the friction between the bottom of the foundation and the ground is sufficient to react against E—that is, when E equals or is less than $P + \beta W$, β being the coefficient of friction. In this case the ultimate resisting moment from passive earth pressure is $1/6\,(bd^3\delta)$. This is case 1 as shown in the stress diagram.

FRICTION

The friction on the two sides of the foundation, which are parallel with the overturning moment M, is

$$2\beta\,(\alpha d^2 c)/2 = \alpha\beta d^2 c,$$

where α is the intensity of active earth pressure at a depth of 1 ft.

The resisting moment from this friction can approximately be estimated to be

$$\alpha\beta d^2\,(c - 3f)\,\sqrt{1/9\,(d^2) + \tfrac{1}{4}\,(c - 3f)^2} + \alpha\beta d^2\,(3f)\,\sqrt{1/9\,(d^2) + 9/4\,(f^2)}$$

Frequently $3f$ will be about $c/2$, in which case the expression is simplified to

$$\alpha\beta d^2 c\sqrt{1/9\,(d^2) + 1/16\,(c^2)}$$

The horizontal component of the active earth pressure on the back side of the foundation will tend to overturn same with a moment of

$$\frac{\alpha d^2 b}{2} \times \frac{d}{3} = \frac{\alpha d^3 b}{6}$$

But the vertical component of earth pressure due to friction acting downward on the foundation will give a resisting moment of

$$\alpha\beta d^2 b\,(c - f)/2$$

The pressure on ground is then increased by the amount of vertical friction on the back, but this being comparatively small, it will in most cases not be necessary to readjust the location of the center of the ground pressure.

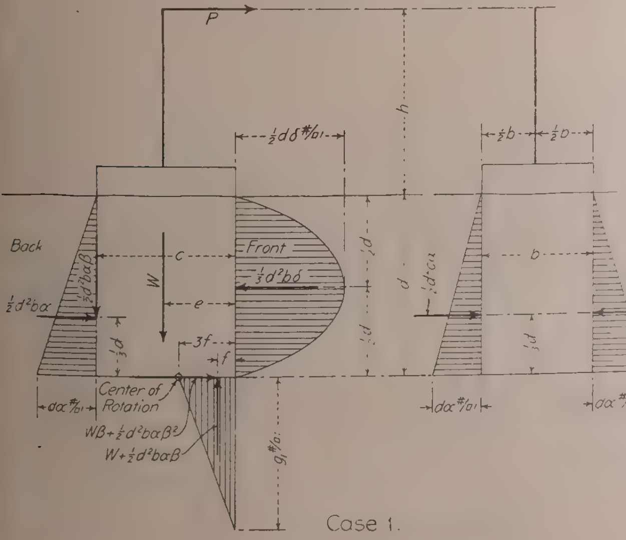
The horizontal component of active earth pressure and the horizontal component of friction on sides of foundation counteract each other and are small compared with E and W. They were therefore left out of consideration when it was stated that E was not greater than $P + \beta W$. The addition of all the ultimate resisting moments should then be somewhat larger than the overturning moment. If this is not the case the dimensions should be increased and the computation repeated.

The various resisting moments can be assumed to act at the same time and to their ultimate extent, on account of the semi-fluid action of the soil, which prevents one of them being completely overcome before the others come into action. What occurs is not a sudden crushing after an infinitesimal deformation but a slow squeezing out of the layers. The sum of the ultimate resisting moment should not be less than the moment which stresses the superstructure to near the elastic limit.

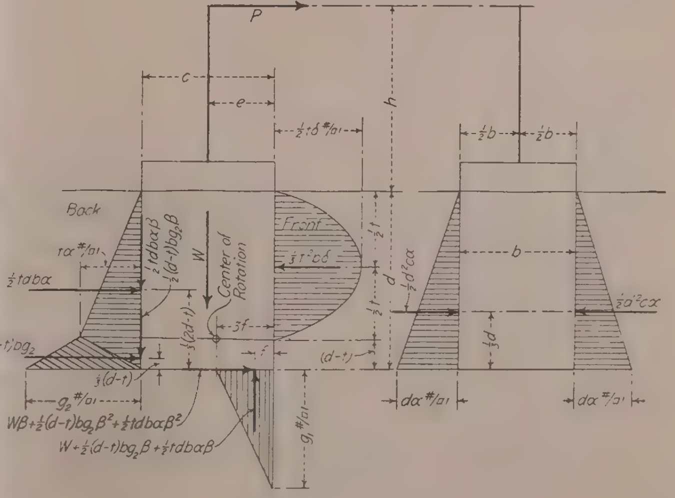
SAFE LOAD

The safe load P should be fixed as a fraction of the overturning load, instead of the working pressure being fixed as a fraction of the ultimate pressure, as there is no straight linear proportion between moments and pressures.

If g_1 is increased to $2g_1$ the resisting moment from the pressure on the bottom of



Case 1.



Case 2.

the foundation will not be doubled but only increased in the proportion $(e-f/2)/(e-f)$. It will be found economical to build the foundation so that $b = c/2 \pm$. For poles carrying power wires the resisting moment from the weight ought to be at least half the total resisting moment to take care of possible digging in the ground near the foundations, or else the factor of safety should be increased.

The value of δ for good soil and a natural slope of 2/3 can be assumed to be 400 lb. per square foot. This is when the friction on the front side of the foundation acting upward on same is left out of consideration. As the movement of the upper parts of the foundation is nearly horizontal when the foundation tips, it is questionable in many cases whether the friction will be developed to any considerable amount. The vertical component of the movement is so slight that it is possible that a fraction of the friction will suffice to compress the earth that amount in a vertical direction. It is then safest not to rely on this friction, which would increase the value of δ very much.

The value 400 lb. per square foot is for foundations where b is about as large as d. When b decreases δ increases. That this must be so can be seen by considering that the foundation when tipping not only forces the earth in front of it up on a slope but also at the same time shears this earth prism out from the adjacent earth, which tries to keep it back by friction. This latter action is independent of the width of the foundation and gains, therefore, in relative value the narrower the foundation is. For $b = 0.5d$, $\delta = 450$; for $b = 0.3d$, $\delta = 500$; for $b = 0.2d$, $\delta = 550$, and for $b = 0.15d$, $\delta = 600$. This is probably why telegraph poles are so much more stable than ordinary figuring would indicate.

CASE 2

It may happen that E is greater than $P + \beta W$. This is the case when the dead load of superstructure and foundation is small, and particularly when there is no foundation, as for ordinary wooden poles set in the ground. The point of rotation will then be raised so that the passive earth pressure on the back of the foundation near the bottom comes into play at the same time the pressure on the front of the foundation is decreased. We then have

$$Et^2/d^2 = P + \beta W + g_2\,(d-t)\,b/2$$

from which equation t can be found. In this case g_2 is the ground-pressure intensity on the back of the foundation near bottom, and can be assumed to be $3g_1/4$.

The resisting moment from the pressure on the front of the foundation is then

$$t^2\,(d-t/2)\,b\delta/3$$

As the horizontal component of the passive earth pressure on the back acts in the same direction as P, it gives an overturning moment of

$$1/6\ \ (d-t)^2 bg_2$$

while the vertical component acts downward on foundation and gives a resisting moment of

$$1/2\ (d-t)\,(c-f)\ bg_2\beta$$

The overturning moment from active earth pressure is

$$1/6\ \ (td)\ \ (2d-t)\ \ b\alpha$$

while the resisting moment of the friction on the back is

$$td\ (c-f)\ b\alpha\beta/2$$

The resisting moment from friction on the sides of the foundation can with sufficient exactness be estimated by drawing a horizontal and a vertical line through the center of rotation and finding the moments of the friction on the four areas, using the distances from the center of rotation to the centers of the pressures as lever arms.

If $3f$ is nearly equal to $c/2$ we get approximately

$$\alpha\beta t^2 c\sqrt{\frac{t^2}{9}+\frac{1}{16}\,c^2}$$

$$+\frac{1}{2}\,\alpha\beta\,(d^2-t^2)\,c\sqrt{(d-t)^2+\frac{c^2}{4}}$$

As this method is somewhat more complicated than where $t = d$, it may be stated that even if E is greater than $P + \beta W$ and t is less than d it does not make any appreciable difference, as long as $0.8E = P + \beta W$, so that the first method can be used as a close enough approximation. But for poles with very narrow and deep foundations t should be found.

For wooden poles without foundations it would give a value approximately 30 per cent too high to figure $t = d$. For wooden poles t can be assumed to be $0.85d$. For wooden poles the approximate formula $M = jbd^3$ is serviceable, where M is the resisting moment, in foot-pounds; b is the diameter of the pole below ground, in feet; d, depth in ground, in feet, and j, weight per cubic foot of soil expressed in pounds. The overturning moment should be taken around the butt of the pole.

Frost-Proof Inclined Skylight

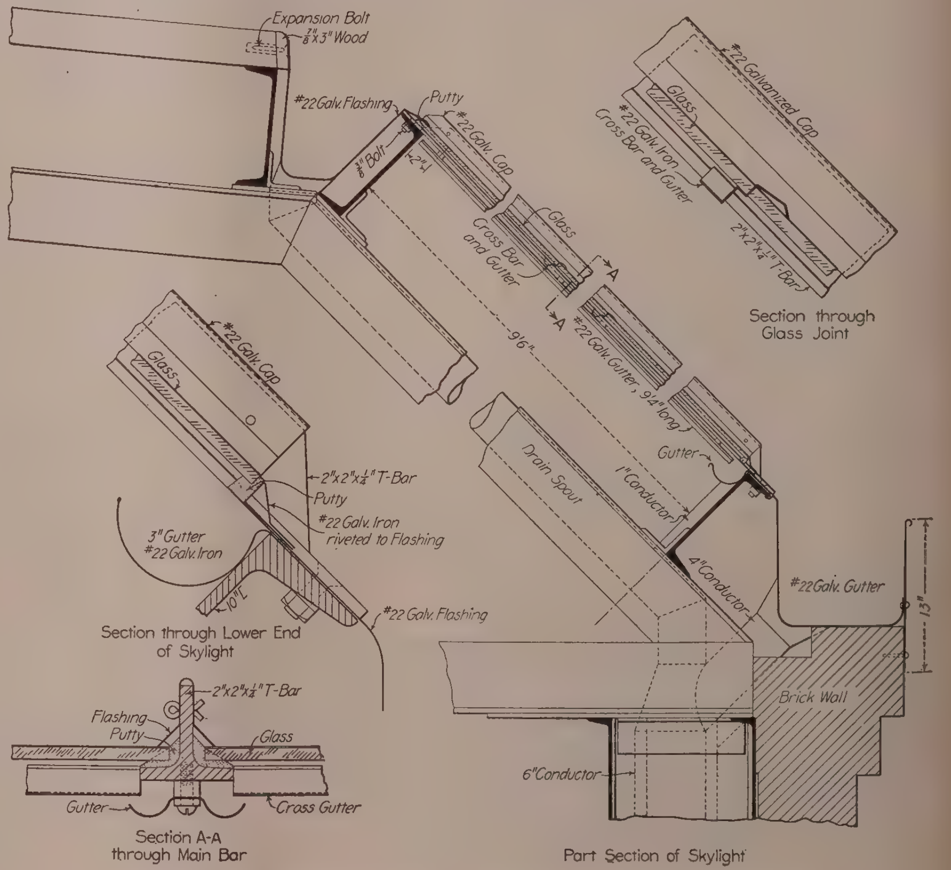
Exposure of Exterior Gutters to Heat of Building Prevents Freezing and Consequent Overflow and Formation of Icicles

THE type of skylight here illustrated has been used for six years on the shop buildings of the American Hoist & Derrick Company, of St. Paul, Minn., and has been applied to a new structural-steel fabricating shop. This building, 90 ft. wide and 200 ft. long, is lighted almost entirely by three full-length lines of these skylights, 30 and 60 ft. apart, which are inclined 45 deg. to the vertical, and are in the planes of the inclined end posts of the roof trusses, 20 ft. apart. The designers consider such skylights much more efficient than windows in the side walls or in the clerestory of the monitor.

Between roof trusses the skylight is carried on top and bottom 10-in. channel purlins which support the galvanized-iron framework and inclined transverse T-bars. The width of the skylight is about 10 ft. over all for one set and 7 ft. over all for another set.

The ribbed-wire glass, ¼ in. thick, has a uniform width of 22 in., and only three lengths are used, so that repairs can easily be made from stock. The glass is supported on inclined T-bar ribs. All joints are made with putty, and the horizontal transverse butt joints are provided with galvanized gutters to carry the condensation to small galvanized gutters under the T-bar ribs. The latter gutters run to larger open gutters parallel to the purlins and are supported on their top flanges, which are drained to interior downspouts. This construction avoids trouble either from condensation or from leaks. The location of all the downspouts inside the building and the exposure of the exterior gutters directly to the interior heat of the building have been found very satisfactory in preventing freezing and the consequent overflow and formation of icicles. For the gutters and flashing for foundry buildings it is recommended that copper should be used instead of galvanized steel.

The skylights were designed under the direction of Oliver Crosby, president and engineer of the American Hoist & Derrick Company, of which H. LeRoy Brink is assistant engineer.



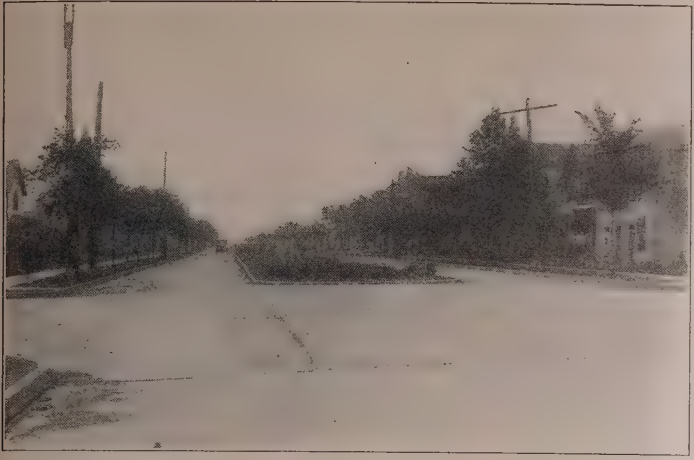
Section through Glass Joint

Section through Lower End of Skylight

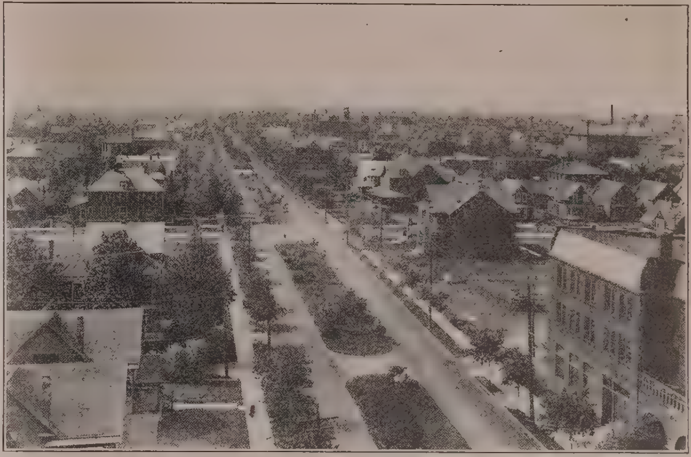
Section A-A through Main Bar

Part Section of Skylight

GUTTER AND GLAZING DETAILS OF FROST-PROOF INCLINED SKYLIGHT



TYPICAL TRANSITION FROM SINGLE TO DOUBLE ROADWAYS



HAMMOND AVENUE PARKED TO REDUCE PAVEMENT WIDTHS

Narrowing Wide Streets in Superior, Wisconsin

Unduly Ample Roadways in Residence Districts Narrowed Down to Reduce Cost of New Pavements

YEARS ago, during the boom days of Superior, Wis., the streets were laid out to care for the traffic of a million people. Many of those in the downtown district were paved with cedar blocks at costs that were almost confiscatory of the adjoining property by the time real-estate values reached normal. When it came time, recently, to repave these streets and many others in residence districts it was found that the space between curbs, as many of them were originally laid out, was entirely unnecessary for the traffic and prohibitory in expense if the whole area were to be covered again.

Tower Avenue, the main artery of the city, has a roadway width of 60 ft. and sidewalks of 20 ft. It has been kept to its original width, and this year several blocks of asphalt with sandstone adjacent to the car tracks have been added to the south end.

Last year Baxter Avenue, a 70-ft. residence street, was paved for a width of 24 ft. with reinforced concrete consisting of a 6-in. layer of 1:2½:5 mixture and a 1½-in. top coat of 1:1:1 mixture. Expansion joints ½ in. wide on top, tapering to ¼ in. at the bottom and 25 ft. apart, were filled with Pioneer asphalt. Recent inspection by a representative of this journal showed the pavement in good shape, although the expansion joints indicated the need of another coat of tar.

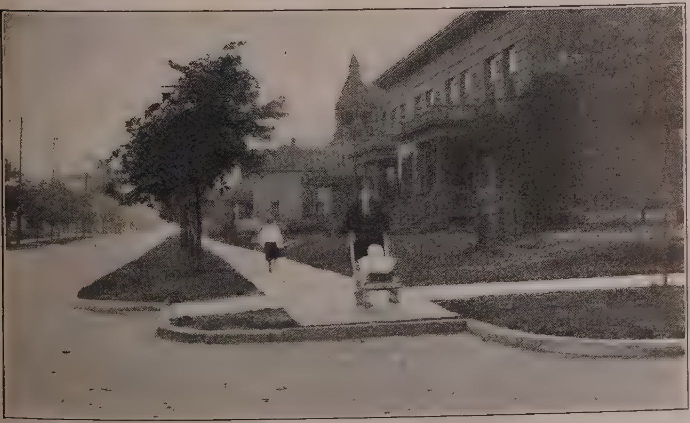
Gas and water mains in this street are laid within 2 ft. of the curb, but on that part of Broadway within the residence section—that is, from Hammond Avenue east—the water main is in the parking 5 ft. back of the curb. At this point Broadway, a 100-ft. street between property lines, is narrowed from a 60-ft. to a 30-ft. asphalt roadway. To reduce the paved area in Hammond Avenue—originally 60 ft. wide between curbs and with a 20-ft. sidewalk space—center parking 24 ft. wide has been adopted between macadam roadways 18 ft. in width.

Grand Avenue, another 100-ft. street, was paved last year with concrete for twelve blocks. Below Sixth Street it is 40 ft. wide and above a 24-ft. parking separates two 18-ft. roadways. Cummings Avenue, formerly a 40-ft. roadway, has been reduced to 30 ft. this year when eighteen blocks were paved with concrete.

In reducing the street widths curbs already in at corners have usually not been disturbed. This was not only to avoid changing catchbasin inlets, but also to give ample turning room. Some of the methods of making the transition are shown in the accompanying photographs.

One of the narrowest pavements put in is at West Third Street, in a residence district where the 500-ft. block ends in a sheer bluff overlooking railroad tracks. A former 44-ft. cedar-block pavement has been replaced by 8-ft. concrete roadways each side of 28-ft. center parking. Turning points exist at the center and inner end of the block. With the 28-ft. width inside of the curbs the street resembles private grounds.

All street work in Superior is done under the general supervision of E. B. Banks, city engineer.



TRIPLE CURVES IN CURBS WITH BOTH STREETS NARROWED

ENTRANCE TO NARROWED ROADWAY FROM A WIDE STREET

Financial Statement on New York's Catskill Aqueduct

EXPENDITURES of $21,000,000 are charged against the New York City Board of Water Supply for 1913, according to the annual report recently issued. Since the organization of the board on June 9, 1905, there has been spent $109,000,000. Construction of the new Catskill supply had cost to Jan. 1, 1914, $79,000,000, and in addition $18,000,000 worth of contracts were then under way. On Jan. 1 the Catskill Aqueduct was 99 per cent completed from Ashokan reservoir to Croton Lake, and from Hill View reservoir to the terminal shafts in Brooklyn the city tunnel was 75 per cent completed. On the nominal date of the report there remained to be awarded, in order to complete the first development of the Catskill supply sufficiently to deliver 250,000,000 gal. of water daily to the five boroughs, contracts to the value of about $7,000,000, nearly all of which were to be begun and prosecuted during the year 1914. It is now possible to make Catskill water partially available at any time, if an emergency should demand it, although the whole aqueduct and its reservoirs will not be completed for some years to come.

A NEW HYDROGRAPHIC CHART of Vineyard Sound and Buzzards Bay, Mass., has just been issued by the U. S. Coast and Geodetic Survey. These waters include the southern approaches to the Cape Cod Canal, the rocks and shoals of which were located last season by the wire-drag party of the Coast Survey.

Water-Discharge Measurement with Chemicals

Titration Method Described by B. F. Groat, Hydraulic Engineer, of Pittsburgh, Before Engineers' Society of Western Pennsylvania—Part II

M. CH. LOUIS, an engineer in Hautes-Pyrenees, has employed the volumetric method with success upon the mountain torrents of that region. It is stated that, in such cases, it is easy to treat solutions containing about one centigram of marine salt per liter without being an experienced chemist.

In applying the method there are various sources of error which must be eliminated before a high degree of precision is attainable. There may be present in the stream, initially, other chemicals which are reduced by the standard, or the dosing chemical may be affected by them.

ELIMINATING ERRORS

It must not be supposed, therefore, that such ratios as p and e can be correctly deduced for practical cases from the atomic weights of the simple elements. In any practical case the chemicals employed for dosing the water, as well as the reagents, are not likely to be chemically pure. Hence these values must be determined from the results of carefully executed tests where precision is necessary, or what is the same thing, methods must be adopted which will eliminate the errors.

A procedure very simple in principle has been suggested by M. Louis. To each sample of exactly 1 liter, add 1 cu. cm. of the bichromate of potassium solution containing 5 grams per liter, shake and add silver nitrate solution until the persistent red color appears. Let it be supposed that v_2 cu. cm. is the volume added, and that q was the rate of dosing at the upper station. Then take q cu. cm. of the dosing solution and dilute with a sufficient volume of the natural river water to cause 1 liter of the resulting solution to react under the same conditions as did the sample taken from below. That is to say, so as to require v_2 cu. cm. of the silver nitrate solution to give the same red color as before. If it be found necessary to add Q liters of the natural river water to get the foregoing reaction, the required discharge will be Q liters per second.

It is not necessary in the dilution test above to employ the same quantities, Q and q, as when dosing the stream. It is only needful to have them relatively the same. Thus, if Q^1 and q^1 are, respectively, the quantity of natural river water and the quantity of dosing solution which have to be mixed to produce the same reaction observed when treating the sample taken from the sampling station we should have $Q : q = Q^1 : q^1$, in which proportion all quantities are known except Q.

If we have accurate subsidiary chemical analyses to determine the values e and e^1 in the cases of the tests of the samples taken from the sampling station and also of the diluted samples made up from the natural river water and the dosing solution, we may write the following equations:

$$Qc_1 + qC = (Q + q)\ c_s ve \qquad (15)$$

$$Q^1c_1 + q^1C = (Q^1 + q^1)\ c_s v^1e^1 \qquad (16)$$

Here v and v^1 are the respective volumes of the standard solution consumed by the samples from the sampling station and the dilution described above. If c_1 be eliminated from these equations, either of the two following equations may be obtained:

$$Q = \frac{qQ^1\ (C - c_s ve)}{q^1(C - c_s\, v^1e^1) + Q^1c_s(ve - v^1e^1)} \qquad (17)$$

$$Q + q = \frac{q\ (Q^1 + q^1)\ (C - c_s v^1e^1)}{q^1(C - c_s\, v^1e^1) + Q^1c_s(ve - v^1e^1)} \qquad (18)$$

If, instead of the accurate analyses for e and e^1, the procedure recommended by M. Louis be adopted, we should have $v = v^1$ and consequently $e = e^1$. Hence $ve = v^1e^1$ and there results

$$Q = Q^1\ (q \div q^1),\ ve = v^1e^1 \qquad (19)$$

$$Q + q = (Q^1 + q^1)\ (q \div q^1),\quad ve = v^1e^1 \qquad (20)$$

as was anticipated above.

In case $c_1 = 0$, equations (15) and (16) furnish at once

$$Q + q = (Q^1 + q^1)\ qv^1e^1 \div q^1ve,\quad c_1 = 0 \qquad (21)$$

TEST COMPARISONS

M. Louis has made the following test comparisons:

Discharges by weir, Bazin formula,
592 : 381 :: 266 : 109
Discharge by chemical method,
592 : 394 :: 270 : 111

In the case of the weir the aggregate was 1348 while in the case of the chemical method it was 1367. The relative discrepancy is, therefore, about 1½ per cent.

The method devised by Dr. Mellet of the University of Lausanne appeals to the writer as the one most likely to eliminate errors and give exact results. It is particularly applicable to turbine testing. A brief synopsis of the method follows:

It has already been explained that chemical reactions may take place subsidiary to those between the principal chemicals present in the solutions treated. In particular, a certain excess of nitrate of silver is necessary to act upon the indicator producing the change in color which attends the end of the reaction. After an extended research concerning the chemistry of very dilute solutions, Dr. Mellet announces that this excess is proportional to: First, the total volume of solution; second, the total quantity of precipitate which remains suspended in the solution for a longer or shorter period of time; third, the total quantity of indicator (potassium chromate) present.

It follows that, if we reduce the samples approximately to the same volume and approximately to the same concentration and at the same time keep the quantity of indicator proportional to the volume of the standard employed the excess of the standard required in any case will be proportional to the total quantity of standard consumed.

If x be the relative amount of the excess required with reference to the total quantity of standard necessary for the titration, then the method described above for treating several solutions will cause x to be a constant ratio in all the titrations, supposing, of course, that the same standard solution be employed in all.

Recurring to formula (1)—printed in last week's issue—it will be seen that if the three concentrations C, c_1 and c_2 can be determined, the discharge Q may be found when the rate, q, of injecting the dosing solution is kept rigidly constant at some previously determined value. It is further clear, that if all three solutions be titrated subject to Dr. Mellet's rules, the excess of silver nitrate required to act upon the indicator in each solution will be proportional to the volume of the standard solution introduced. If v_0, v_1, v_2, respectively, be the volumes of the standard required per liter of the three solutions having the concentrations C, c_1, c_2, then we shall have $C = v_0\ (1 - x)$, $c_1 = v_1\ (1 - x)$, $c_2 = v_2\ (1 - x)$.

If these values of the concentrations be substituted in formulas (1) and (2) we shall have, after cancelling the common factor, $(1 - x)$

$$Q = [(v_0 - v_2)/(v_2 - v_1)]\ q \qquad (22)$$

or

$$Q + q = [(v_0 - v_1)/(v_2 - v_1)]\ q \qquad (23)$$

These formulas show that, by adopting Dr. Mellet's method, we have eliminated all quantities except the volumes of the standard solution consumed in the three titrations and the rate of injecting the dosing solution. In short, it is not even necessary, by this method, to know the concentration of any of the solutions including that of the standard. Neither is it necessary to determine, or to subtract from the volumes of the standard, the excess of silver nitrate required to act upon the indicator, this having been eliminated by cancellation.

RESULTS OF ACTUAL TEST

In order to illustrate the process, Dr. Mellet has given the results of an actual test upon a turbine of the Day power plant at Vallorbe, as follows:

The dosing solution was introduced into the turbine at the rate of $q = 0.1211$ liters per second. This solution and the normal feed water of the turbine were sampled before the test. At the end of the sixth and ninth minutes of the test samples were taken from the tailrace.

Ten cubic centimeters of the dosing solution were diluted to 1 liter and 10 cu. cm. of this liter taken for titration. It required 35.5 cu. cm. of silver solution, or $v_0 = 355\,000$.

One liter of the normal feed water was reduced by evaporation to 10 cu. cm. and this required 3.8 cu. cm. of the silver solution or $v_1 = 3.8$.

Dr. Mellet determines the size of the tailrace samples so that when they are reduced by evaporation to 10 cu. cm. they will each contain approximately the same quantity of dosing chemical as did the 10 cu. cm. from the diluted liter of dosing solution which was, conveniently, 1/10000 of the quantity of chemical in one liter of dosing solution. If the discharge of the turbine is roughly estimated, and each tailrace sample be of such a size that it contains as many cu. cm as there are liters per second of discharge, and if the rate of injecting dosing chemical be about 1 decaliter per second, the tailrace samples will be properly portioned in accord with the principles explained.

A rough estimate of the discharge of the turbine indicated that it was from 100 to 300 liters per second. Hence, following Dr. Mellet, the tailrace samples should contain from 100 to 300 cu. cm. each.

Sample 1, taken at the end of the sixth minute contained 200 cu. cm. and, when evaporated to 10 cu. cm., required 33.5 cu. cm. of the silver solution, which corresponds, therefore, to 167.5 cu. cm. per liter, or $v_2 = 167.5$.

Sample 2, taken at the end of the ninth minute, contained 150 cu. cm. and, when evaporated to 10 cu. cm., required 25.15 cu. cm. of silver solution, or, $v_2 = 167.67$.

These values might now be substituted in formula (22) or formula (13), according to the circumstances of the test, but Dr. Mellet makes the calculation very simple and with sufficient exactness as follows:

$$\text{Sample 1.} \quad Q = 0.1211 \ \frac{355000}{167.5 - 3.8} - 0.1211 = 262.4965 \text{ liters per sec.}$$

$$\text{Sample 2.} \quad Q = 0.1211 \ \frac{355000}{167.67 - 3.8} - 0.1211 = 262.2241 \text{ liters per sec.}$$

These values differ by about 0.1 per cent, which is about the limit of error mentioned by Dr. Mellet for such a case.

In order to carry out such a test successfully, one should be thoroughly acquainted with the details of the process as set forth in Dr. Mellet's valuable paper. However, the following directions for conducting the titrations and evaporations may be of interest to the engineer:

The dosing solution is diluted in such a manner that 10 cu. cm. will contain several centigrams of the salt. One liter of the normal feed water is evaporated in a porcelain casserole 10 cm. in diameter, into which the liquid is poured by degrees as fast as it evaporates. This evaporation to 10 cu. cm. takes place in a bain-marie.

The sample from the tailrace or lower sampling station is evaporated in the same manner to approximately the same volume, the volume of the sample being so taken that we shall have several centigrams of the salt to titrate.

The titration takes place in a porcelain casserole (for the solutions evaporated, in the same casserole which has served for the evaporation). To 10 cu. cm. of the liquid to be titrated first add two drops of the chromate solution, then introduce slowly the silver solution from the burette. If the proper tint does not appear after the addition of 10 cu. cm. of this solution add a drop of the chromate and continue the titration, and so on until the color does change. This titration takes place beside a second casserole of the same size, containing the liquid for comparison. This latter is prepared at the moment needed, with 10 cu. cm. of chloride of sodium solution, two drops of chromate and 10 cu. cm. of silver solution.

MIXING SOLUTION WITH WATER

One of the most important questions for the engineer to determine is the method for effecting a perfect mixture of the dosing solution with the water of the stream to be gaged. In the case of mountain torrents the question is comparatively clear. In the case of broad sluggish rivers it is quite the reverse. It is also easy to see that, in the case of turbine testing, there may or may not be serious difficulties. In the case of a single runner discharging into a long tailrace, where there is ample opportunity for the mixture to be effected, it would seem that no serious trouble should be anticipated. In the case of a single one of several units taking water from a single flume, each being driven by several runners connected to the same shaft, it may prove very difficult to secure the same degree of concentration for all of the runners of the unit being tested.

The course recommended by the writer in the latter case is to make use of current meters in a preliminary test to determine the distribution of velocities in the forebay of the unit to be tested. By subdividing the cross-section into a sufficient number of partial areas of equal discharge and by dividing the uniformly flowing stream of dosing chemical equally among these partial areas, the rapidity of mixing may be greatly accelerated. A further aid is to employ a centrifugal pump for performing a part of the mixing. The pump takes its water from the forebay above the point where its discharge is equally divided among the partial areas of equal discharge. The dosing chemical is fed to the pump through a pipe connected to the suction. It is clear that the pump must run at a perfectly uniform rate and that every effort must be made to have its discharge equally divided among the partial areas. The dosing chemical must be fed at a uniform rate as in all other cases. All the details of this method will depend upon circumstances and success will follow only upon the exercise of good judgment and substantial engineering skill.

If the discharge, Q, equations (2) and (23), is practically constant, it must follow that $Q + q$ is practically constant also, since q has been assumed constant. In this case q need not be maintained rigidly constant so long as it is small relatively to Q. Thus, if Q is 50 000 liters per second and q is 5 liters per second, of a concentration of 250 grams per liter, then q may be altered to 10 liters per second without altering $(Q + q)$ by more than $5 \div 50\,000$; that is by more than 1/100 per cent, which is negligible in tests of 1/10 per cent precision.

Therefore, in the case considered, q may vary between the limits 5 and 10 liters per second, that is by 100 per cent, without affecting the results, provided an average sample from the tailrace be taken and the simultaneous total volume of salt solution injected during the test be observed.

Thus in the case of constant, or nearly constant, turbine discharge, the constant head device may be omitted. But a continuous sample should be taken from the tailrace at a practically constant rate. The total volume of salt solution consumed may then be read from a calibrated scale on the side of the salt-solution tank while a liter of the continuous sample, or samples, from the tail race furnishes the average sample or samples, as the case may be.

INTRODUCING SOLUTION

It would seem that the best method of introducing the strong salt solution would be by means of a system of perforated pipes in the head-race connected directly to the salt-solution tank, while the continuous tailrace sample could be pumped up from a similar system of much smaller, say ¼-in. perforated pipes. A large centrifugal pump might be employed, as already described, while a smaller one agitates the strong salt solution or pumps the dissolving water through the dissolving box before beginning the test.

The latter remarks apply particularly to large discharges. For example: If Q = 50 000 liters per second it would require, at 0.03 grams per liter, 1.5 kilograms of salt per second. For one hour's test it would take $3600 \times 1.5 = 5400$ kilograms, or nearly 6 tons of salt. If fifty tests averaging, say, 4 tons of salt are to be run, it would require 200 tons of salt, which would cost upward of $200, freight and handling not included. If the tests can be shortened in duration a corresponding saving can be effected.

New York's First Convict Camp

State Highway Department Is Establishing Well-Equipped Living Quarters in Catskill Mountains for Sing Sing Honor Men

THE New York State Highway Department is completing quarters for a gang of convicts from the prison at Ossining who are to spend three seasons locating a 4½-mi. road near Palenville, in the Catskill Mountains. The establishment of this community, the first of its kind in the State, was noted in the Current News pages of this journal July 26 and is described at length in the July issue of the New York "Highway News." Fifty men are now housed at the camp, and plans which include a sewerage system and a 1/3-mi. water line are going forward. It is expected that eventually 200 men, including the officials, will be stationed at this point.

The conditions of the project lend themselves readily to the use of prison labor. The highway starts at Palenville, about 14 mi. south of Catskill, and skirts the south side of Kaaterskill Creek on a shelf which at times is 175 ft. above the bed of the creek. Rock cuts to reduce the 25-per cent grades will make the work slow and costly. The Legislature has appropriated $190,000 for this project. The road will, in fact, form a direct route over the hitherto almost impassable barrier between the Hudson River and the interior towns of Greene County.

Three large buildings—the guards' living quarters, and a dormitory and a mess room for the workers—have been built at an isolated point about 3 mi. from Palenville, where a relocation of the road will necessitate heavy work. The camp is directly at the foot of a hill below Sunset Rock on Kaaterskill Mountain, surrounded by South Mountain and North Mountain, and it is necessary to travel a mile and a half to reach another habitation.

In addition to the large buildings there have been erected a well-equipped bath and dressing room for the men. Water for the engines and for cooking and drinking purposes will be secured by tapping a spring on Round Top Mountain, 3600 ft. high. It will be necessary to run 1600 ft. of 2-in. pipe down the mountain, crossing the creek above high-water level and proceeding underground to a 4000-gal. tank immediately back of the bath house. The steep gradient will allow a pressure of approximately 20 lb.

Besides establishing this comfortable home for the unique community, the highway department has had to purchase an expensive road-construction equipment. This includes a 5-ton traveling crane, a traveling derrick and a 20-hp boiler, three rock drills, a 48-hp portable boiler, a blacksmith outfit, four dump cars, a 30-hole blasting battery, a 3-ton, double-chain screw hoist and 4 tons of rail for tracks.

BULK SHIPMENTS of Universal Portland Cement totaled 43,114 bbl. in June. This is almost three times the total bulk shipment of the same brand of cement during 1912.

Testing Concrete Aggregates

Discussion by Nathan C. Johnson of C. M. Chapman's Paper Before American Society for Testing Materials

SINCE the paper "Test for Concrete Aggregates" (see Engineering Record, July 11, page 39) was presented at the July convention of the American Society for Testing Materials the following written discussion has been submitted by Nathan C. Johnson, of the Raymond Concrete Pile Company:

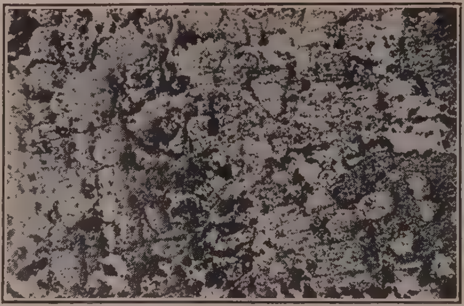
To say that the strength of concrete depends primarily upon the strength of the materials entering into it seems almost unnecessary. It is doubtful, however, if we are customarily giving this obvious fact its due importance or remembering it in selecting materials for our everyday concrete. It is also trite and almost unnecessary to say that the concrete which in density most nearly approaches homogeneous natural stone of proper strength and character is the best concrete for any given requirements. For instance, if a certain load is to be borne, concrete of a density equal to that of a natural stone, the normal strength of which would be capable of withstanding the given stress, would be the most economical concrete to use.

STRENGTH OF STONE

We determine the strength of a natural stone by stressing to the breaking point a piece of the stone of definite cross-sectional area. It is easily determined that the strength per unit area is a certain amount, say 18,000 lb. per square inch. The determination of this unit strength is simple, for we are dealing with a homogeneous material. Further, this unit strength is inherent in and evenly distributed throughout the mass—i.e., the strength of the cube expresses the total cohesive strength of all the tiny particles which, ages ago, united to form the rock mass of which the cube was a part.

It must, therefore, be true that the fragments of the broken cube would possess inherently the same strength per unit area as the original mass; and following the reasoning a step further, if it were possible to replace these fragments in their original relation one to the other and to hold them in this relation by some cementitious material, each layer of this material being infinitely thin and of a strength equal to the adhesion or cohesion between the tiny particles that formed the stone, we could impose a load on the re-formed cube equal to that borne by the original cube. If such a replacement and cementing were possible we should have an ideal concrete.

Indeed, some stones are, of themselves, natural and ideal concretes. If we magnify a pebble sufficiently we render visible its component grains. It is possible that we may also see the cementing layer between these grains, though as we do not always know what to look for we may not recognize it. The accompanying micro-photographs illustrate such natural concretes. One is a sandstone, almost so soft as to be shaly. A high magnification was required to bring out this structure. The other is a micaceous sandstone wherein the component aggregates—tiny, but none the less true aggregates—and their boundaries, with possibly also the cementitious material between them, may be more easily distinguished. These are true concretes in miniature and a tantalizing ideal.



MICACEOUS SANDSTONE

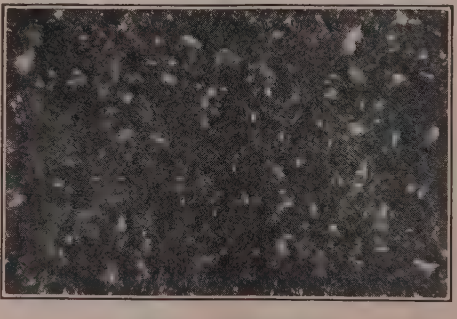
In practice, however, the ideal cannot be reached. We may have all the fragments of the crushed cube, but neither their perfect rearrangement nor the infinitely thin and strong cementitious layer can be had. We are limited in both respects, but to overcome the first limitation we add to the fragments cementitious material in excess, diluted with sand.

These conditions in no wise change the inherent strength of our stone fragments. The true measure of their unit strength remains the unit strength of the mass from which they were broken, and if properly bedded they are capable of exerting this strength in concrete. To determine their actual strength it remains, then, to pack them together in a relation similar to that which they would hold in a commercial, everyday concrete, to keep them in that relation and to bed them thoroughly in a matrix having a strength at least equal to the inherent strength of the fragments. In such an arrangement the crushing strength of the mass must closely approximate the inherent strength of the stone.

Mr. Chapman's method of aggregate test would seem to offer a nearer approach to the ideal than any in present use. There are, however, some considerations which seem to have been overlooked.

SELECTION OF MATRIX

The first is a question as to the obtaining of a matrix sufficiently strong to allow the full working stress of the stone to be developed. For the sake of eliminating as many variables as possible, let us assume that neat cement grout is used to fill the voids in the stone, as sand would introduce new and indeterminate strengths in the matrix. And in this connection further difficulties may be encountered, as a neat cement grout sufficiently thin to enter the voids might shrink unduly. But assuming the end desired to be accomplished with a normal paste and all voids filled, we are confronted with the fact that unless the crushing strength of the stone is less than 14,000 lb. per square inch, which is a generous figure for the crushing strength of normal, neat cement of reasonable age, the test specimen will fail by crushing the matrix, rather than by crushing the stone. And corollary to this is the second consideration that if the matrix crushes, the adjacent fragments, or pieces of gravel having point contact, may crush below their actual strength, as stress values at contacts would momentarily rise extremely high, even with low unit loads on the specimen.



SOFT SANDSTONE

COMPRESSIVE STRENGTH OF STONE

Illustrative of this point the following compressive strengths of stones of various kinds are interesting as compared with the figure before taken as that of the compressive strength of neat cement—i.e., 14,000 lb. per square inch.

COMPRESSIVE STRENGTH OF STONE

Kind of rock	Average crushing strength 2-in. Cube edge	2-in. Cube bed	3-in. Cube edge	3-in. Cube bed
Gneiss (Chester, Pa.)	6,097	5,446	9,505	6,426
Gneiss (Germantown, Pa.)	19,891	15,555	11,636	13,984
Granite (French Creek, Pa.)	19,997	14,348	17,274	7,910
Mica Schist (Conshohocken, Pa.)	20,038	15,680	10,417	7,532
Sandstone (Curwensville, Pa.)	10,218	8,013	7,513	4,463
Quartzite (Lumberville, Pa.)	..	..	14,841	8,637
Limestone	..	..	..	..
Marble	..	13,000	..	..

Sound trap rock, 4-in. cubes, 33,300 lbs. per sq. in.
Seamy trap rock, 4-in, cubes, 19,400 lbs. per sq. in.

From the above results it would seem that Mr. Chapman's method might give erroneous results with the great majority of aggregates commonly used, as the strength of the matrix would probably be below that of the aggregate under test.

It would be interesting also to compare the results as given on Mr. Chapman's curves with the values of the coarse aggregate in bulk. A tentative comparison of trap rock as aggregate and in bulk may be made from Mr. Chapman's curve and from the figures before cited. As aggregate, the trap rock shows a strength at 28 days of about 2500 lb. per square inch. In bulk, the trap rock tested at the Watertown Arsenal has a strength of 33,300 lb. per square inch maximum and 19,400 lb. minimum. It would seem as if these figures tend to confirm the correctness of the reasoning that indicates the test as being one for matrix rather than for aggregate.

The varying strengths with different aggregates shown on Mr. Chapman's curves may result from greater or less voids, with consequent thicker or thinner matrix layers between the fragments of aggregate. It would be both interesting and instructive to illustrate these points further with other photo-micrographs that the writer has in his possession, but a more detailed presentation of this subject is beyond the province of the present discussion.

It would seem that, with slight modification, Mr. Chapman's method could be made to give very accurate results; and it is needless to say that some standard method of testing large aggregate is badly needed. If, instead of neat cement, or a cement grout of ungraded sand as a filler between the pieces of large aggregate, a standard grout of strong, carefully graded sand, preferably crushed quartz, or crushed trap, were used, in quantity equal to voids, a definite percentage, a matrix of definite thickness of layer and of strength equal to the inherent strength of the aggregate would be secured. With such a matrix, properly aged, concordant results closely approximating the true strength of the aggregate should be obtained.

CONCRETE PEDESTALS FOR SUPPORTING STEEL COLUMNS OF DOCK

Foundations for Largest Ore-Shipping Dock in the World

Concrete-Capped Piles, with a Mile of Steel Sheet Piling Inclosing a Sand Fill, Supporting Ore Bins Holding 112,000 Tons

FOUNDATIONS for the fifth and latest ore-shipping dock of the Duluth, Missabe & Northern Railway Company at Duluth, Minn., are 2146½ ft. long, 64 ft. wide and contain 24,225 cu. yd. of concrete. The dock is the largest of its kind in the world and has a storage capacity sufficient to load fourteen vessels with an 8000-ton cargo. On the inside of the dock the concrete capping the 14,600 round piles is 4½ ft. thick, while at the outside faces it is 7½ ft. thick, increasing in height by three 1-ft. steps. The piles, which are spaced on centers 2½ ft. apart transversely and 3 ft. apart longitudinally, are driven into the clay about 40 ft. Between the two outer rows of round piles U. S. steel interlocking sheet piles 40 ft. long were driven 6 in. below low water around the entire dock to retain sand pumped in from the bay to within 18 in. of low-water mark. On the row of piles outside of the steel sheeting are 12 x 16-in. fir timber caps tied across the dock with 1¼-in. rods spaced on 12-ft. centers to hold the steel piles in place. The round piles were cut off 6 in. below low water so as to be saturated at all times. These piles were all driven from the ice and the sheet piles from scow drivers. Steel columns for the dock rest on 395 concrete pedestals or piers 5 ft. square on top and spaced on 12-ft. centers.

CONCRETE PLANTS

Two concrete-handling plants were used mainly because it was anticipated that winter would come on before the first plant could complete the work. The first was mounted on double flange wheels running over rails 23 ft. apart laid on 8 x 12-in. crossties, 26 ft. long, supported by capped piles. This track was removable, and as its support was under water relaying consisted in floating the section passed over around the plant to the piles ahead, no attempt being made to sink the track until the plant was pulled onto it. Sections were 36 ft. in length, as expansion joints were specified on the plans every 144 ft.

The procedure was to pour concrete from the 100-ft. boom of the 55-ft. Insley tower over the heads of the workmen setting up forms into the section nearest the completed work.

Sand and stone were furnished by one 300-yd. and three 100-yd. scows. The aggregate was reclaimed from the scows by a 1½-yd. Hayward clamshell bucket and dumped into a 25-yd. stone bin, or 15-yd. sand bin elevated above the No. 13 Smith mixer. The stiffleg derrick operating the clamshell was mounted on a separate traveling platform just ahead of the mixer outfit. The two platforms were fastened together so as to minimize boom work.

Cement for a two days' supply, 1000 bbl., was stored on a barge alongside the mixing plant, from which it was wheeled by warehouse trucks and raised 10 ft. to a feeding platform by an ordinary wheelbarrow elevator.

During one three-hour period 180 cu. yd. were placed, but during a ten-hour day the best run was 485 cu. yd. This plant placed 1966½ ft. of the total length of 2416½ ft. of concrete-covered piling.

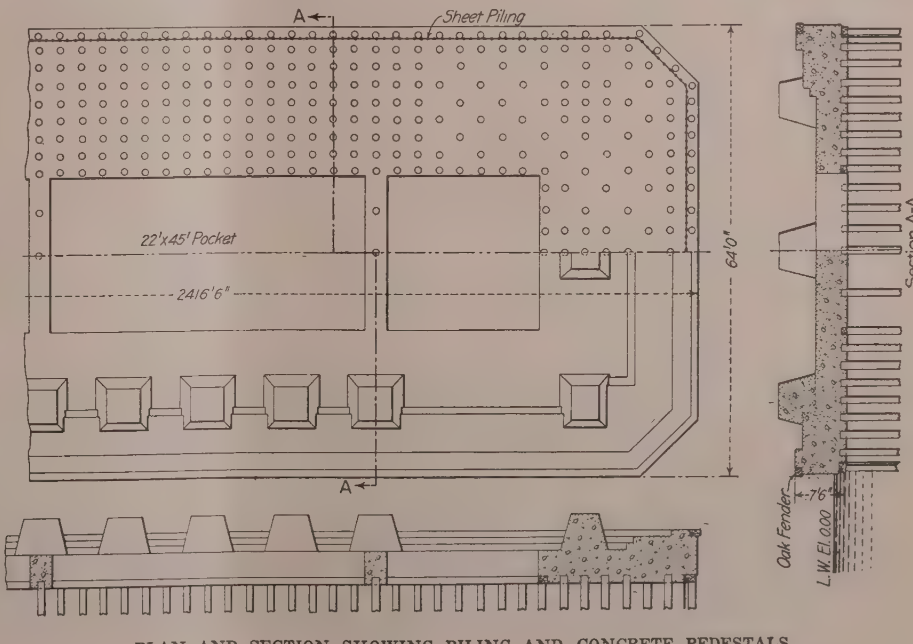
The second plant, installed to place 450 lin. ft., was stationary and erected on completed work. It was equipped with an 18-cu. ft. mixer, with a similar arrangement for handling aggregate as that of plant 1, but the mixed concrete was raised to a storage hopper from which ¾-yd. Lakewood radial gate cars were filled and then pushed by hand over one of two industrial tracks to the forms. It was the intention to lay a crossover at the outer end of these tracks, but it was found unnecessary to do this, the shuttle service with four cars proving sufficient.

From the central raised platform on which the tracks were laid the concrete was distributed by short chutes, the outer end of which was manipulated by a floating stationary boom. During the latter part of the season the concrete was covered with about 6 in. of sand as a protection against frost.

REINFORCING MATTRESS

Reinforcement was placed 6 in. above the top of the piles and consisted of ¾-in. round bars spaced on 6-in. centers longitudinally and 12-in. centers transversely. In the outer faces ½-in. rods were placed on 12-in. centers two ways. The bars were made into mattresses, 20 ft. wide and 36 ft. long, on a raft. Two 12 x 12-in. timbers for handling were then fastened over the top of the mat lengthwise and secured to the cross-bars at intervals of 2 ft. to prevent slipping. When in place the wires were cut and the timbers returned to the raft.

An overhang of 4 in. beyond the longitudinal stringers under the outer edge of the concrete required the forms to be supported on cantilevers. These consisted of square 1-in. bars 18 in. long, spaced 2 ft. apart and bent up at the end for 1 in. The bars were spiked to the longitudinal



PLAN AND SECTION SHOWING PILING AND CONCRETE PEDESTALS stringer and supported a 2 x 6-in. plank laid parallel with the stringer to carry the Blaw forms or wooden sheets.

Placing these form supports and running the tie wires for the sheets back to spikes in the tops of piles, all under the ice-cold water, was a trying experience for the men, and various expedients were used to handle the under water work. For driving spikes a "gun" was used consisting of a piece of 2-in. pipe in which a 1¼-in. steel rod with an up-set end worked. For those spikes driven to provide anchors for the form ties, a loop of wire long enough to extend above water was passed around the head and the head end of the spike then drawn into the pipe, which had a slot at the lower end to receive the wire. Thus the spike could be driven without entering the water and subsequently attachment could be made to the loop, all operations being above water.

PLACING FORMS IN WATER

Concreting was begun in August of last year and completed in November. Work on the foundation was started in February.

The Bates & Rogers Construction Company, for whom C. S. Smith is superintendent of construction, was the contractor for the pier foundations. The American Bridge Company erected the superstructure for the Duluth, Missabe & Northern Railroad Company, for which H. L. Dresser is chief engineer.

Seros Hydroelectric Development in Spain

Part I—Building 80,000-Horsepower Generating Station with Concrete-Lined Canal System Over 14 Miles in Length in Record Time of Fifteen Months

By J. C. STEVENS

Formerly Project Engineer, Ebro Irrigation & Power Company, Ltd., Now of Portland, Oregon

THE Barcelona Traction, Light & Power Company, Ltd., through its subsidiary, the Ebro Irrigation & Power Company, Ltd., is developing three hydroelectric projects on Rio Segre and its principal tributary, Rio Noguera Pallaresa, in Spain. The first is the Gerri project. This was originally started as a temporary plant of 3000 hp for construction purposes only, but before completion the plans were altered to make it a permanent station of 4000 hp. This plant has been in operation since February, 1913. The second project comprises the Talarn Dam and Pallaresa Canal. This unit involves the storage of 170,000 acre-feet of water in a river valley near Talarn by the construction of a concrete dam 282 ft. in height. At this dam will be a power house from which the tailwater will be carried in an 11-mi. canal to a second power house. The Talarn Dam was 10 per cent completed on May 1, 1914. The third project is the Seros, near Lerida.

The present article will deal with the Seros project, which was completed in April, 1914. The relation of the Seros plant to the general geography of the country is shown on the accompanying map. The power generated will be largely used in Barcelona, 87.6 mi. distant, and outlying districts.

The project consists of a diversion dam and intake structures to pass a flood of 157,000 sec.-ft. and divert 2120 sec.-ft. for power purposes; an upper canal 11.8 mi. in length, concrete-lined throughout to the regulating reservoirs; regulating reservoirs created by seven earth dams and giving an available daily storage of 2230 acre-feet; a lower canal, 2.62 mi. in length exclusive of reservoirs, with a capacity of 4240 sec.-ft.; forebay and penstocks, power house and transformer station, where four 16,000-hp turbines are installed (with provision for a fifth unit) and where the voltage is raised from 6000 to 110,000 volts for transmission to Barcelona, and a tailrace that returns the waters to the river. The total length from diversion dam to tailrace is 17.6 mi. The total natural fall in the river between these points at low stage is 170.6 ft.

The general arrangement of the dam, intake, racks, sluice gates and canal gates is shown in one of the drawings. As the river carries considerable gravel at time of flood, three large sluice gates are placed in such a manner as to scour out accumulations of débris in front of the racks. These gates are 26.25 x 18.7 ft., with a clear lift of 22.97 ft. They are of the Stoney type, hand-operated, manufactured by Ransomes Rapier, Ltd., of London.

The floor of the discharge channel in front of the sluices is of creosoted pine 6 in. thick, held in place by 18-in. I-beams partly embedded in concrete. The planks lie in the direction of the flow of water and are so arranged as to be readily replaced when worn out. The removable racks, thirty in number, are placed in fifteen bays. The rack bars are a special type spaced 5.9 in. apart in panels 6.56 ft. wide. A traveling differential chain block is used to elevate them for cleaning or repairs.

The six intake gates, also of the Stoney type, are 11.15 ft. x 19.68 ft., with a lift of 12.47 ft., and are to be operated by hand. The bottom of the intake forebay has an inclination of 1.5 per cent toward the canal gates. In front of these gates is a sand trap and a sluice gate to return to the river the débris that may pass the racks.

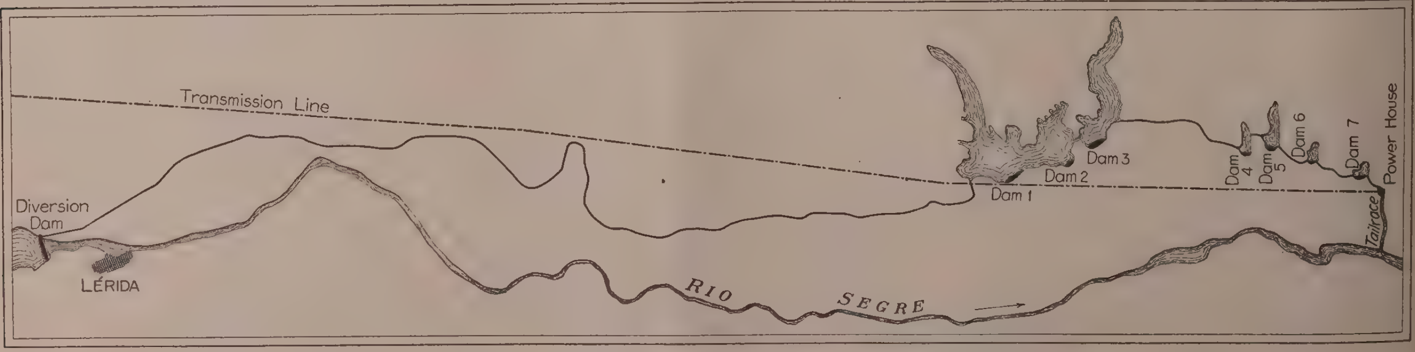
OVERFLOW DAM

The dam is of the ogee type. The foundation is a fairly hard sandstone of varying thickness. Immediately below this sandstone is a stratum of indurated sandy shale varying from 5 to 15 ft.

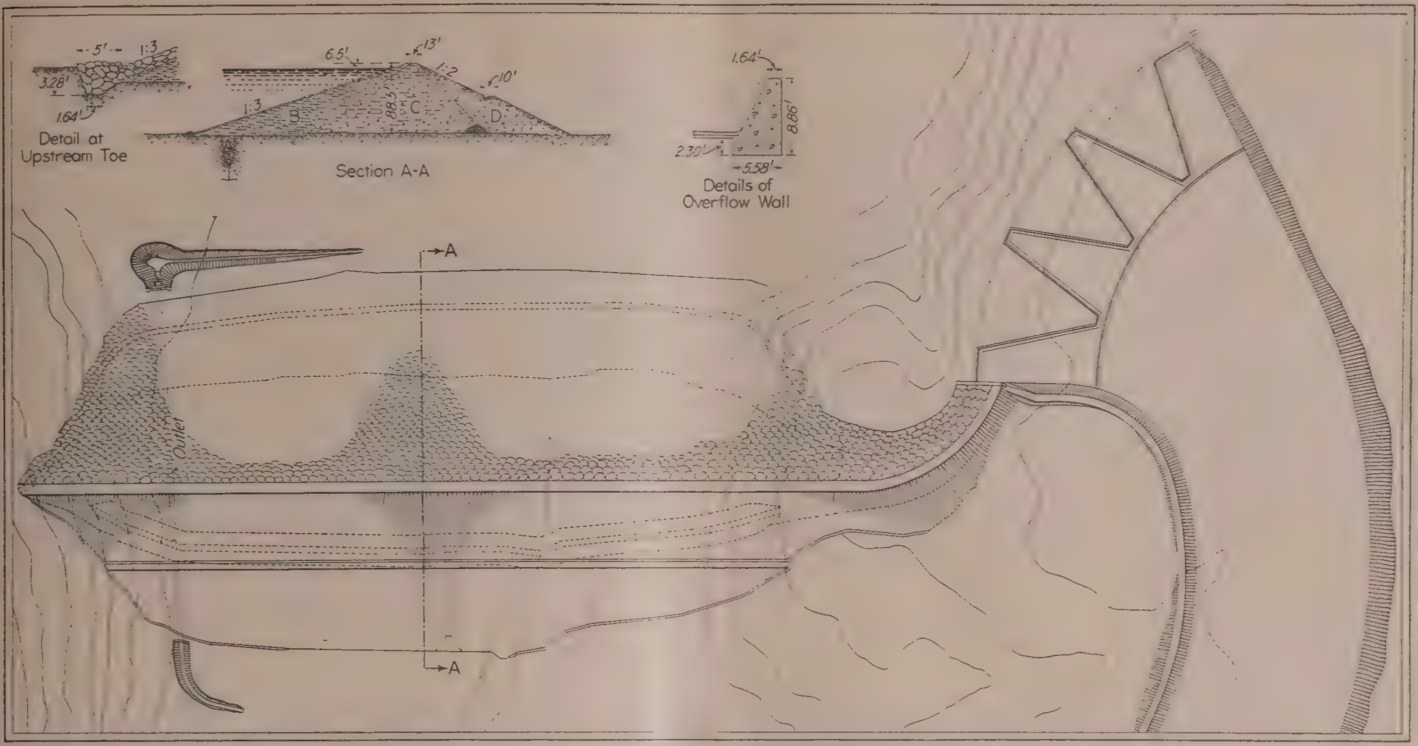
No apron was built below the dam, as it was decided to wait until the effect of overflowing water on the river bed could be studied. If necessary, a protecting apron can be easily added at a later date.

During construction the plan was first tried of driving a row of steel sheet piling in line with the upstream face in order to reduce excavation and serve as a cutoff, but the plan was not successful and it was abandoned. The piling did not reach solid material, but was bent and doubled so badly by the small gravel that most of it had to be removed by hand before the excavation could be completed. No serious difficulty was encountered in excavating by the ordinary methods in open trench and unwatering by motor-driven centrifugal pumps. The river was diverted by timber and stone cofferdams to alternate sides of the channel and finally through the large sluice gates while the closure was made.

The total length of the overflow proper is 1289.7 ft. and of the logway opening 14.8 ft. There is a low buttress between



LAYOUT OF HYDRAULIC FEATURES OF SEROS HYDROELECTRIC DEVELOPMENT ALONG RIO SEGRE IN SPAIN



GENERAL LAYOUT AND DETAILS OF DAM 3, SHOWING DESIGN OF ZIGZAG SPILLWAY AND LOCATION OF SEEPAGE DRAIN

them, 11.5 ft. thick, making a total length between abutments of 1316 ft. The dam is capable of passing a maximum of 132,500 sec.-ft. over the ogee section, 23,000 sec.-ft. through the sluice gates and logway and 2100 sec.-ft. into the canal or 157,600 sec.-ft. in all.

REGULATING RESERVOIRS

About 3 mi. above the power house the canal line crosses a series of low valleys, seven of which were closed by earth dams to create four regulating reservoirs. The arrangement of these is shown in the general plan. The table gives general quantitative data for each of the earth dams.

One of the drawings shows a plan and cross-section of dam 3, the largest of the group. The same general design is followed for the remaining six. The materials designated by letters in the cross-section are:

Class A—Puddle refill of the cutoff trench. It is composed of equal parts of clay and bank gravel in which all stones over 1 in. in diameter were rejected. The clay and gravel were mixed by hand at the side of the trench and shoveled into it, thoroughly mixed with water and compacted by tampers.

EARTH DAMS FOR REGULATING RESERVOIRS

Dam	Max. height, ft.	Max. width of base, ft.	Length of crest, ft.	Embankment, cu. yd.	Riprap, sq. yd.
1....	57	312	1,273	270,400	19,000
2....	41	226	886	58,890	6,400
3....	75	436	1,312	497,250	26,400
4....	33	184	510	41,808	5,230
5....	40	213	682	75,634	8,000
6....	30	164	351	42,510	4,230
7....	47	236	781	59,800	2,270
Total				1,046,292	71,530

Class B—Natural bank clay free from stones larger than 2 in. diameter spread in 8-in. layers inclined at a slope of 3 per cent toward the upstream face of the dam, sprinkled and rolled.

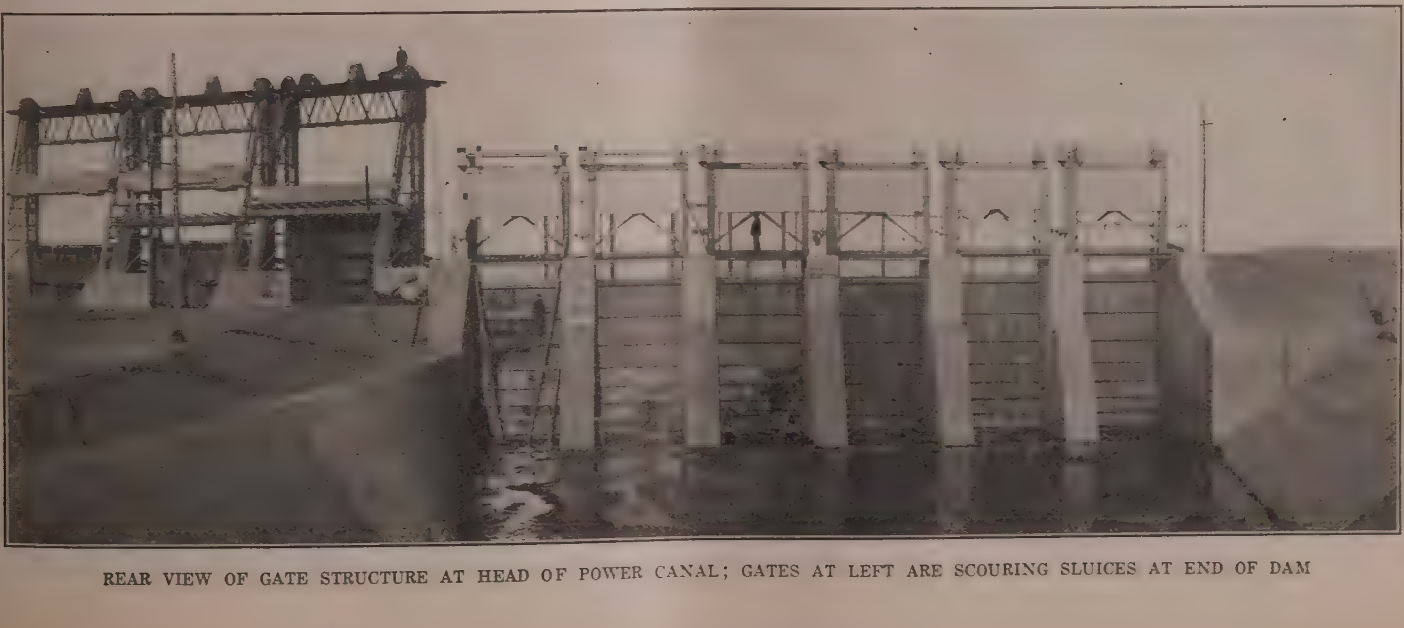
Class C—Borrow run of earth or clay and gravel of coarser material than Class B. In placing it the finer and more compact material was placed in the upstream portion. To accomplish this, section *C* was divided into four equal parts and the limiting sizes and quantity of stones were fixed in each quarter in such a manner as to secure as far as practicable a gradual decrease in the imperviousness from the front to the back faces of this section without sacrifice of density.

CLASSES OF MATERIAL

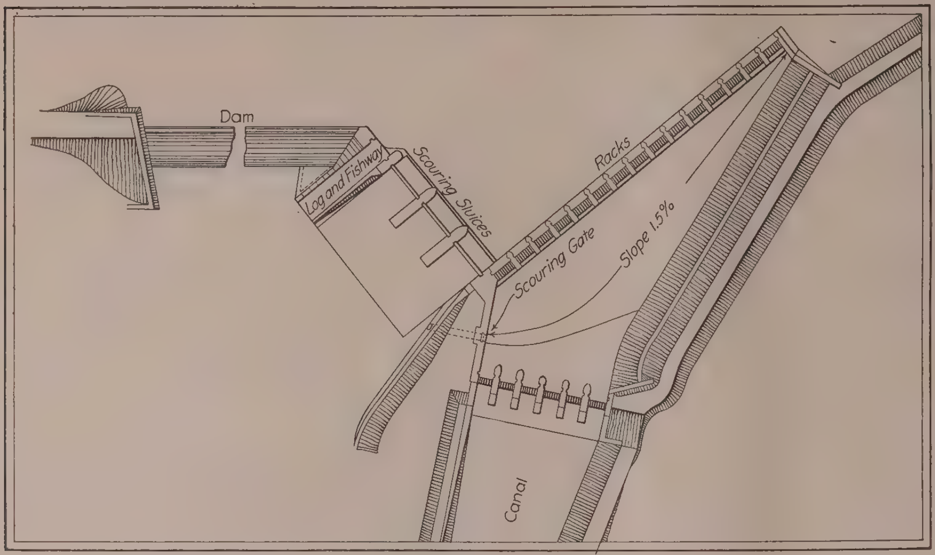
Class D—Borrow run of coarse material of earth and gravel with larger stones intermingled and spread in layers of about 30 in., with only sufficient rehandling to insure a dense mass and at the same time one which was pervious to water filtration.

Class F—A rock dike built transversely across the dam near the lower face to act as a blind drain to relieve any filtration that may penetrate the face of the dams.

The embankment for dams 1, 2 and 3 was made largely from borrow pits excavated by a Bucyrus steam shovel No. 60 and by Austin standard graders, the for-



REAR VIEW OF GATE STRUCTURE AT HEAD OF POWER CANAL; GATES AT LEFT ARE SCOURING SLUICES AT END OF DAM



PLAN OF DIVERSION DAM AND RACKS AND GATES AT INTAKE OF POWER CANAL

mer working in borrow pits on the hillsides above the water line of the reservoir and the latter in the beds of the "barrancos" upstream from the dams. The remaining dams were made by slips, wheel scrapers, carts or Koppel cars in a manner to suit the local conditions. About one-half of the total earth embankments were taken from the canal section.

The downstream face of dam 1 was planted with Bermuda grass in order to induce a vegetable growth to prevent furrowing of the sides during rains and to prevent wind erosion.

EMBANKMENT ROLLERS

Steam rollers were utilized on dams 1, 2 and 3, but on the others a smaller roller, drawn by mules, was used. This roller was made of a series of cast-iron disks or wheels, with faces about 1½ in. wide. They were fixed on parallel axles, and interlocked to keep the wheels from clogging with mud. The bed could be loaded with stone to any desired weight.

The cutoff trench on dams 1 and 2 was excavated about 13 ft. deep in the bed of the valley and then a row of steel sheet piling was driven to refusal in the bottom of the trench. Some of the piles penetrated to a depth of 40 ft. The trench was then backfilled with puddle, inclosing the upper end of the steel sheeting.

On the canyon sides the cutoff trenches were refilled with puddle as the dams were raised. Where the trenches cut the horizontal rock strata of the canyon sides keys were built into the rock, forming a seepage cutoff. The foundations of all dams were stripped of all vegetable matter and the portion under section *B* was excavated 1½ ft. as a minimum to insure a good bond with the original ground.

Outlet pipes were laid in trenches excavated in the original ground at the sides of the canyons under dams 1, 2 and 3. These pipes were 3.28 ft. in diameter, made of steel plate 7/32 in. thick, riveted in 20 ft. lengths. They were joined by heavy cast-iron flanges bolted together. The trench was then refilled with concrete, leaving a minimum thickness of 8 in. around the pipe. A large cutoff collar surrounds the pipe at the point where it crosses the puddle trench and smaller collars project from the concrete at each joint of the pipe.

Above the concrete the trench was refilled with clay and gravel puddle, which joined with the earth embankment of the dam.

At the upper end of the pipes are a reducer section and a 31.5 in. gate valve operated by a shaft, with the mechanism on top of a valve tower.

SPILLWAYS

The seven dams virtually form four reservoirs. The first one is formed by dams 1, 2 and 3. The connection between dams 2 and 3 is an unlined channel 131.2 ft. wide on the base with side slopes roughly 1:2, the bottom being level. The second reservoir is formed by dams 4 and 5, the connection between them being an unlined canal with a bottom width of 59 ft. and side slopes 1:1. The third reservoir is formed by dam 6 and the fourth by dam 7.

No spillways are provided for the second, third and fourth reservoirs, but a spillway at the left end of dam 3 will provide a discharge capacity of 10,000 sec.-ft. under a head of 2.25 ft. This spillway is of the zigzag type, 997 ft. long, and operates entirely on freeboard. A channel with a capacity of 10,600 sec.-ft. discharges the water into the canyon below the dam. This spillway will not come into action except in the case of an unprecedented flood on the drainage area above the dam, coincident with the stoppage of the power canal. The power canal has sufficient capacity for all known floods.

(To be concluded)

Snoqualmie Tunnel Headings Meet

THE headings of Snoqualmie Tunnel, the 11,900-ft. single-track bore the Chicago, Milwaukee & St. Paul Railway is driving through the summit of the Cascade Mountains about 60 mi. east of Seattle, Wash., met on Aug. 4, but owing to a slide that blocked the heading about 1800 ft. from the east portal, it was necessary to hole through from the west heading. As the east is a top heading and the west a bottom one, it was not until Aug. 18 that the lines could be cleared for final checking.

Final checking of lines, grade and distance, according to information received from C. F. Loweth, chief engineer, was as follows: The west line fell 0.15 ft. north of the east line; elevations checked to 0.13 ft., and the actual tunnel distance was 6 ft. longer than the surface lines. Line and grade meeting as they did are considered very satisfactory, and while 6 ft. in distance appears to be considerable, no especial effort was made to get this measurement to a refinement, as at each time it was made there was from 12 to 14 ft. of snow over the summit.

Growth in Chicago Sanitary Bureau

THE steady growth of the activities of the Chicago Sanitary Bureau is pointed out in the August bulletin of the Chicago School of Sanitary Instruction. According to the bulletin, a total of 13,751 complaints has been received for the first seven months of this year, as against 10,022 for the corresponding period last year. There were 1471 building plans, of which 1230 were for new buildings approved for July, as against 1110 for the same month last year, and a total of 8723 for the seven months' period this year, as against 7715 for the seven months' period last year.

As indicating progress made in securing better indoor conditions as to air and proper ventilation, the ventilation division of the bureau reports that, for the seven months of this year, out of 974 inspections made it was found necessary to serve notices in only 52 of the cases inspected, while for last year for the same seven months the division made 10,057 inspections.

The total number of suits brought by the sanitary bureau for the seven months' period of this year was 1464, of which 1304 have been disposed of; for the same period last year 694 suits were brought and 794 disposed of, the excess being suits that were standing over from the previous year.

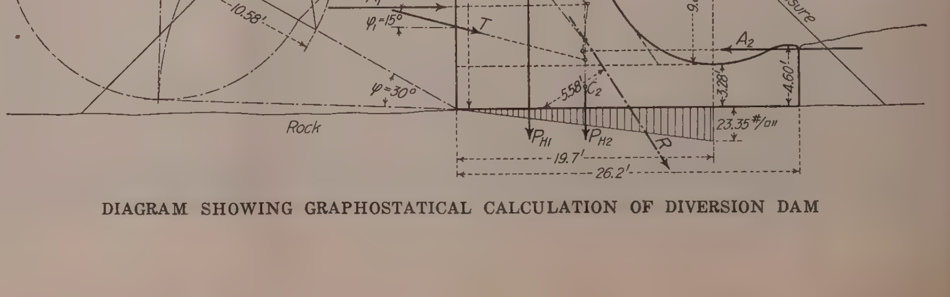


DIAGRAM SHOWING GRAPHOSTATICAL CALCULATION OF DIVERSION DAM

VIEWS OF CANAL TERMINAL BULKHEADS SHOWING TYPE OF CONCRETE CONSTRUCTION AND ARRANGEMENT OF FENDER TIMBERS

Barge Canal Terminals

Anchored Bulkhead Walls and Other Novel Features of Design Adopted at Various Points along the New York Waterway

By J. A. O'CONNOR
Terminal Engineer, Department of New York State Engineer and Surveyor

PROBABLY the chief obstacle in the way of the general use of the old Erie Canal has been lack of proper terminal facilities for the general public. Docking and transfer charges at private wharves have been so heavy as to be practically prohibitive. Attempts to remedy this condition have been fruitless, with the result that the tonnage carried on the Erie Canal to-day is about half the amount carried between 1880 and 1890.

The improvement of the canal system of the State of New York by the construction of the Barge Canal at a cost of more than $100,000,000 has raised the vital question of providing adequate terminal docks which would be controlled by the public. After an exhaustive study by the Terminal Commission appointed by Governor Charles E. Hughes the proposition was submitted to a vote of the people, with the result that in 1911 $19,800,000 was appropriated for the construction of terminal docks and warehouses along the line of the canal.

The design and construction of these docks have brought forth a number of interesting problems for the engineer, since in nearly every case construction must be carried on in or under water. Several types of docks have been designed to meet the conditions found at the various locations. Descriptions, together with drawings and photographs illustrating the various types of dock walls which have been used, follow. Up to the present time contracts for docks and harbors only have been let. Warehouses and freight-handling machinery will be designed and put under contract later.

ALBANY

At the site of the terminal dock at Albany an old timber bulkhead, in a badly deteriorated condition, existed along the 1500 ft. of riverfront to be occupied by the proposed terminal dock. In constructing the new work it was necessary to confine the face line to the adopted bulkhead line as established by the United States Government. This line extended only a few feet outside of the old dock, the top of which was uniformly about 13 ft. above the water level. Therefore, to construct a gravity wall it would have been necessary to excavate the old dock and a considerable amount of earth behind it—an expensive operation. After careful consideration a type of reinforced-concrete sheet-pile bulkhead, with heavy concrete and steel ties anchored in concrete piles driven back of the old dock, was adopted.

The anchorage for the bulkhead was obtained by driving at 25-ft. intervals two Raymond concrete piles about 20 ft. long and capped with a large block of concrete, the front bearing surface of which measured 12 ft. long by 4 ft. vertically. From this anchor several large steel reinforcing rods incased in concrete extended to, and were anchored into, that portion of the front wall cast upon the concrete sheet piles which formed the new bulkhead below the water line. These piles were 12 in. thick and 24 in. broad, with an average length of about 26 ft. When driven the piles extended about 6 ft. above the water surface, 12 ft. through the water to the river bottom, and 8 ft. below into the river bed. The piles were reinforced with four $\frac{7}{8}$-in. steel rods placed on the front side and four $\frac{3}{4}$-in. rods on the back, hooped with $\frac{1}{2}$-in. rods not more than 12 in. apart lengthwise of the pile. In addition, each pile had a male and a female section of interlocking steel piling cast in either side, which after driving united each pile to the adjacent ones, thereby forming a tight and continuous concrete bulkhead below the water line. After the piles were driven, the top 2 ft. of the steel rods were exposed and a light concrete wall was constructed on top. This top wall was 8 ft. high and was heavily reinforced with steel rods. The steel from the top of the piles, together with the steel extending from the anchors through the ties, was all interwoven with the wall steel, and after pouring the concrete the several parts of the bulkhead were all tied together in the monolithic mass of stone and cement. In addition to the sheet piles a square reinforced-concrete bearing pile was driven every 6 ft. along the face of the dock and about 2 ft. in front of the sheet piles.

The construction of the Albany terminal dock is now practically complete. No serious difficulties were encountered, although pockets of boulders interfered somewhat with the driving of the sheet piling and prevented their being anchored to rock. Details of the wall are shown.

TROY

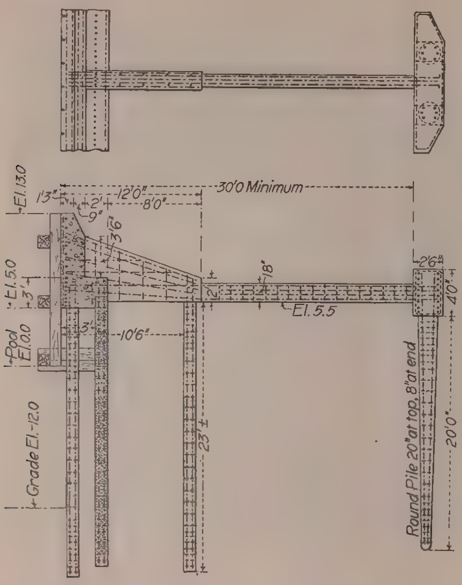
The site of the Troy terminal is the easterly bank of the Hudson River, between Liberty and Adams Streets. It is in the immediate vicinity of the warehouse district and adjacent to the waterfront terminals of the Boston & Maine and the New York Central & Hudson River Railroads. The dock wall is to be constructed on the present bulkhead line. An old timber dock which has fallen into decay must be removed to allow the construction of the new concrete wall, which is to be about 960 ft. in length.

Rock is encountered at an elevation slightly above the proposed grade line of the river channel, which indicates at once that the dock wall should be a concrete retaining wall of the gravity type, with its footing on rock.

The wall itself is a standard type and has no features which would be of special interest. Near the north end of the dock however, it is proposed to construct a movable gangway, which will provide an easy means of unloading small boats which would have difficulty in hoisting freight over the top of a dock wall 15 ft. above mean high tide. Hoisting machinery for this gangway will be mounted on steel towers and provided with hand-operating machinery.

Three large sewers, two circular and one egg-shaped, will be encountered in the construction of this terminal. The portions of these which will be cut away in connection with the construction of the dock wall will be reconstructed as reinforced-concrete circular sewers.

After the completion of the contract, which provides for the building of the dock wall, there will be constructed a large storehouse at the northerly end of the terminal site which will have track connection with the adjacent railroad terminals. It is also proposed to provide freight-handling machinery which will transfer freight directly between the railroad freight cars and barges, as well as between barges and the



BULKHEAD AND PILE ANCHORAGE

freight house. Plans for this feature of the improvement have not yet been made.

The contract for the terminal dock and gangway has been awarded. Sections of the dock wall are being built within steel-sheet pile cofferdams.

SCHENECTADY

The site of the terminal at Schenectady is adjacent to Washington Avenue, near the end of Water Street, and embraces a portion of Van Slyke Island and of an old channel of the Mohawk River known as the Binne Kill. The dock wall, which is now in process of construction, is 1058 ft. in length. The plans provide for a terminal basin 200 ft. wide to be excavated in front of the wall and connected with the Barge Canal through the Binne Kill.

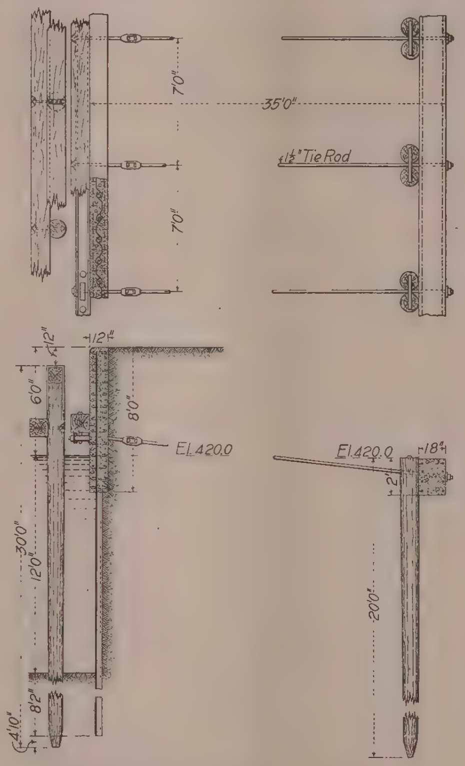
In designing the dock wall several types were considered, but the nature of the foundation material encountered favored the construction of a concrete wall and slab resting on wooden piles. Van Slyke Island is an alluvial deposit. The soil is a light loam, with very little bearing power and is very soft when wet. The line of the dock wall crosses the Binne Kill channel at the south end, extends across a corner of the island and terminates near the middle of the channel at its northerly end. This location of the wall and the nature of the material encountered render it practically impossible to construct a wall which would have to be anchored back to prevent overturning.

The slab is 2 ft. thick and is reinforced top and bottom with expanded metal. It rests directly on wooden piles spaced 5 ft. apart along the wall and 4 ft. transversely. A line of batter piles is driven along the back of the slab to furnish additional stability against sliding. The two rows of piles at the front are spaced 2 ft. 6 in. apart longitudinally in order to carry the additional weight of the concrete dock wall. The wall itself is 7 ft. high and is of the gravity type. The slope of the earth beneath the slab is protected with a 3-ft. layer of riprap.

Two large sewers were encountered in the construction of this terminal. The flow from the larger of these is to be taken care of by a reinforced-concrete box sewer resting on a pile foundation. The second sewer is a 20-in. circular pipe emptying into the Binne Kill at an elevation approximately 5 ft. below the normal stage of water. In order to avoid expensive cofferdam construction it was decided to provide for the flow from this sewer in a rather novel manner. Two lines of wooden sheet piling are to be driven so as to form a 2-ft. clear channel between them. This channel is to be covered with a wooden flooring, fastened to the tops of the lines of sheet piling, which are to be driven to the water surface at low stage. This wooden-box sewer will be carried along the channel of the Binne Kill at about the contour which passes through the outlet of the present sewer, and will terminate immediately below the dike near the north end of the wall.

The Schenectady terminal is in process of construction. A considerable portion of the concrete slab has been built and dredges are pumping the material from the basin into the Binne Kill behind the wall.

The conditions at Rome were such that the successful driving of concrete piles was somewhat problematical and uncertain on account of the hard, gravel foundation. At this place it was proposed to construct 900 ft. of dock parallel to and only a short distance from the canal channel. To construct a gravity wall in deep excavation would have required an enormous amount of pumping on account of the gravelly formation and probably almost unlimited amount of seepage from the adjacent canal. Finally it was decided to excavate a trench and drive a continuous line of heavy interlocking steel piling to a point several feet below canal grade. The top several feet of this piling will be incased in a narrow wall of concrete and the entire bulkhead fastened with large steel tie rods, spaced every 6 ft. lengthwise of the wall and anchored to a continuous line of wooden piles driven parallel to the bulkhead, 40 ft. back. After the completion of the sheet-steel bulkhead the material in front will be removed by dredges.



FENDER DETAILS AND TIE RODS

All the steel piling has been successfully driven and capped by the concrete wall. The accompanying drawings show the design.

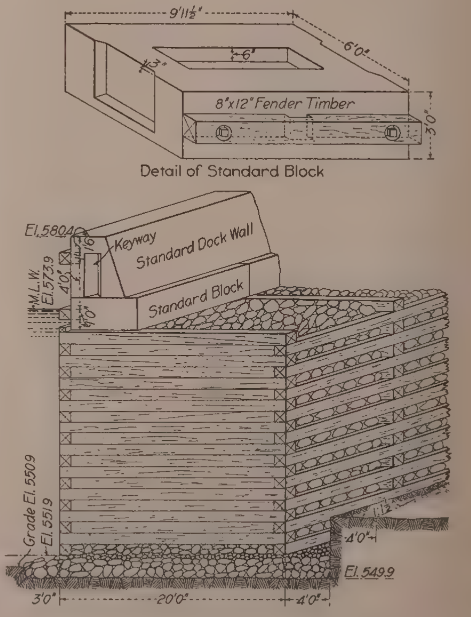
Erie Basin, the western terminus of the Erie Canal, is a harbor formed by the construction of the New York State breakwater in Lake Erie, near the foot of Genesee Street, Buffalo. From this old breakwater three piers extend out into the basin, which is obstructed by a large number of sunken wrecks.

The plan for improving this harbor for Barge Canal terminal purposes contemplates the removal of the southerly end of the breakwater and all of the old piers. All wrecks will be removed and a depth of water of 23 ft. will be provided in the basin. Two new piers 400 and 600 ft. in length, respectively, will be constructed in the vicinity of Genesee and Wilkeson Streets. A channel 280 ft. wide and 13 ft. deep, bounded on one side by an industrial pier and on the other by 300 ft. of dock wall, will provide dock space and an ample entrance for canal boats into the Erie Canal.

Local conditions of open water and storms on Lake Erie necessitate the use of forms of construction which may be put in place without the use of cofferdams or other protection. The piers and adjacent deep-water dock walls will be built of rock-filled timber cribs on which will rest a continuous concrete dock wall 8.5 ft. high. The body of each pier between the cribs which outline it will be built of material dredged from the harbor. Ample means for tying up boats will be supplied by mooring cleats, spaced at frequent intervals along the dock wall.

The site of the dock wall provided for canal boats, near the entrance of slip 3, lies wholly on land. In order to avoid excessive excavation and provide an economical form of dock it was decided to build this portion of the dock wall of steel sheet piling, capped with a concrete wall 8 ft. high and 12 in. thick. This wall is to be tied back to a reinforced-concrete anchorage beam 30 ft. back of and parallel to the wall. Wooden piles are to be driven directly in front of the anchorage beam at the points where it is intersected by the tie rods.

The contractor has begun work on this contract. Several wrecks and a large amount of earth excavation have been removed from the harbor. The accompanying drawings show the type of construction which is to be used in this terminal.



CRIB AND STANDARD CONCRETE BLOCK

Factors Affecting Structural Timber

Discussion of Quality of Wood with Reference to Strength, Durability, Density, Grain, Moisture, Sapwood, Knots, Checks and Shakes

By H. S. BETTS
Administrative Assistant, U. S. Forest Products Laboratory, Madison, Wisconsin

THE two qualities of greatest value in the wood of structural timber are strength and durability. In order properly to select material for a given use a knowledge of the influence of the various physical characteristics of timber on its strength and durability becomes necessary. The characteristics which may be considered in judging the properties of timber are density of wood, direction of grain, moisture condition, proportion of sapwood, and, in addition, defects such as knots, checks, and shakes.

DENSITY

The strength, hardness, shock-resisting ability, and stiffness of wood vary with its weight in small, clear, straight-grained pieces of the same moisture content (see diagram, Plate 1, and list of species in the table). What then is needed is some means of estimating density from a visual inspection. Wood is made up of concentric rings of growth. Each ring consists of two parts, springwood and summerwood. The springwood, as its name indicates, is formed first, and in the coniferous woods commonly used for structural purposes is the lighter colored, softer part of the ring. The summerwood is built up later in the season and is darker and heavier than the springwood (Plate 2). In certain woods, notably the southern pines, the change from springwood to summerwood is distinctly marked, so that the proportion of summerwood in an annual ring or in a cross-section can be closely estimated (Plate 3). Summerwood is considerably heavier than springwood. A number of tests on small pieces of summerwood and springwood cut from wide-ringed loblolly pine showed a density and strength about twice as great in the summerwood as in the springwood. The proportion of summerwood in a cross-section of a stick, therefore, becomes a means of estimating its density and strength. It should be remembered that density is the basic factor governing strength and that the proportion of summerwood as a satisfactory indicator of strength is dependent on the difference in density of the two parts of the annual ring and on the closeness with which these parts can be differentiated.

It is to be regretted that very little data are available in regard to the influence of density on durability in the case of any one species. The heavier species are not necessarily the most durable, since some of the lightest woods, as cedar, redwood, and cypress, have excellent lasting properties, while some of the heavier hardwoods, as hickory and red oak, are notably less decay-resisting. On the other hand, the lighter woods, basswood and cottonwood, are not durable, while the heavy woods, osage orange and locust, are durable.

As to the resin content of the pines being a means of judging their durability, there are apparently little data available on this point also. While the toxic properties of

SPECIES USED IN PLATE 1, WITH CORRESPONDING LOCALITY OF GROWTH AND REFERENCE NUMBERS

Hardwoods

Species	Locality	Ref. No.
Ash, black	Wisconsin	29
Ash, green	Louisiana	35
Ash, white	Arkansas	40
Basswood	Wisconsin	4
Beech	Indiana	42
Birch, yellow	Wisconsin	38
Elm, rock	Wisconsin	50
Elm, slippery	Indiana	37
Elm, white	Wisconsin	23
Gum, cotton	Louisiana	30
Gum, red	Missouri	24
Hackberry	Indiana	33
Hickory, big shellbark	Mississippi	54
Hickory, big shellbark	Ohio	67
Hickory, bitter nut	Ohio	59
Hickory, mocker nut	Mississippi	56
Hickory, mocker nut	Pennsylvania	65
Hickory, mocker nut	West Virginia	66
Hickory, nutmeg	Mississippi	43
Hickory, pignut	Mississippi	60
Hickory, pignut	Ohio	68
Hickory, pignut	Pennsylvania	70
Hickory, pignut	West Virginia	69
Hickory, shagbark	Mississippi	57
Hickory, shagbark	Ohio	63
Hickory, shagbark	Pennsylvania	58
Hickory, shagbark	West Virginia	64
Hickory, water	Mississippi	61
Locust, honey	Indiana	71
Maple, red	Wisconsin	34
Maple, sugar	Indiana	39
Maple, sugar	Wisconsin	49
Oak, post	Arkansas	51
Oak, red	Arkansas	47
Oak, red	Indiana	46
Oak, red	Louisiana	45
Oak, swamp white	Indiana	62
Oak, tanbark	California	44
Oak, white	Arkansas	52
Oak, white	Indiana	55
Oak, white	Louisiana	53
Oak, yellow	Arkansas	48
Oak, yellow	Wisconsin	41
Osage orange	Indiana	72
Sycamore	Indiana	27

Conifers

Species	Locality	Ref. No.
Cedar, incense	California	12
Cedar, white	Wisconsin	1
Cypress, bald	Louisiana	26
Douglas fir	Wyoming	21
Fir, amabilis	Oregon	18
Fir, Alpine	Colorado	3
Fir, white	California	9
Hemlock	Wisconsin	8
Hemlock, western	Washington	22
Larch, western	Washington	28
Pine, lodgepole	Colorado	15
Pine, lodgepole	Wyoming	16
Pine, longleaf	Louisiana	36
Pine, Norway	Wisconsin	25
Pine, shortleaf	Arkansas	31
Pine, sugar	California	11
Pine, western yellow	Arizona	10
Pine, western yellow	California	17
Pine, western yellow	Colorado	19
Pine, white	Wisconsin	13
Redwood	Humboldt Co, Cal.	7
Redwood	Mendocino Co., Cal.	14
Spruce, Engelmann	Grand Co., Colo.	6
Spruce, Engelmann	San Miguel Co., Colo.	2
Spruce, red	New Hampshire	20
Spruce, white	New Hampshire	5
Tamarack	Wisconsin	32

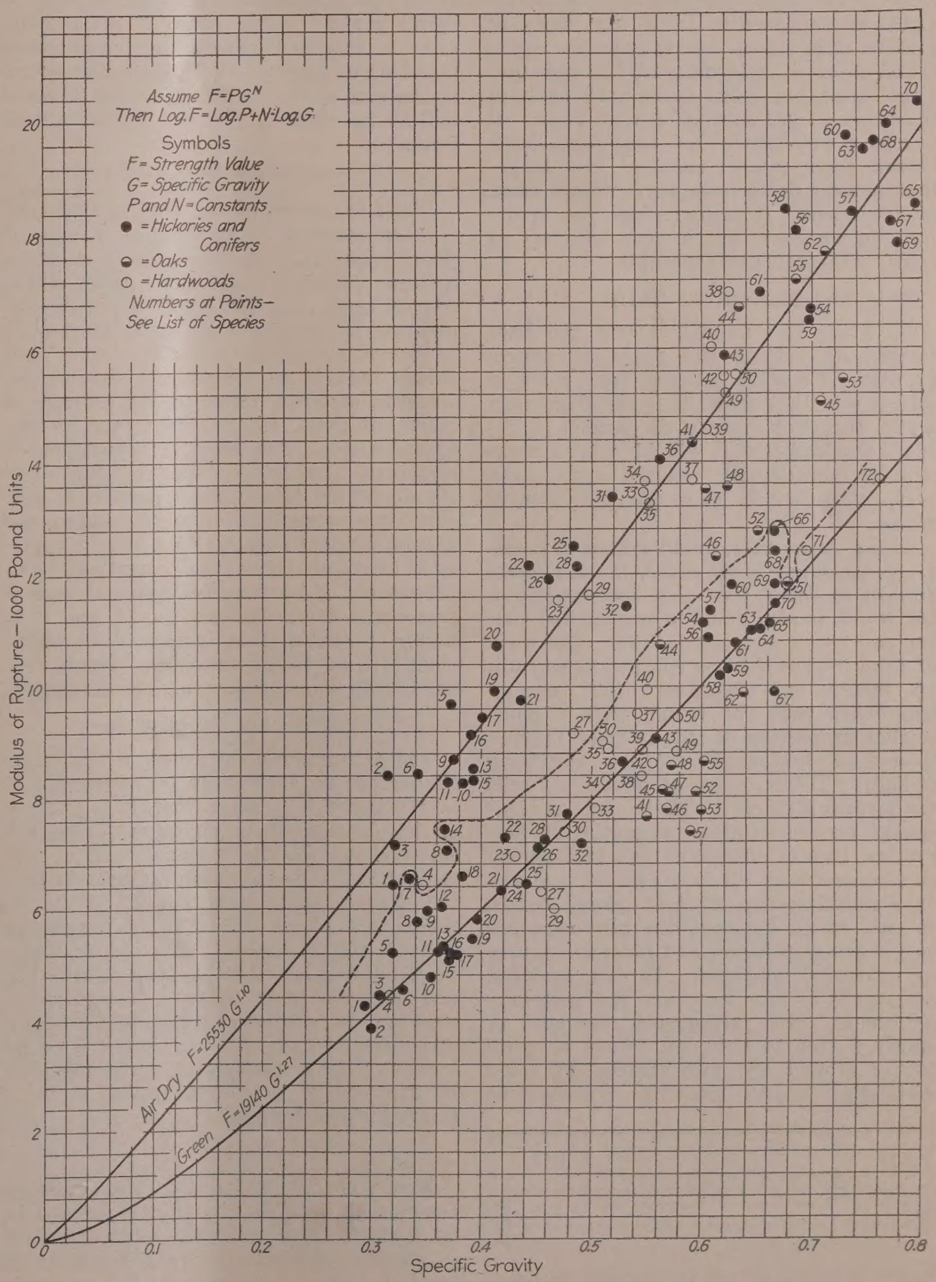


PLATE 1—RELATION OF MODULUS OF RUPTURE TO SPECIFIC GRAVITY

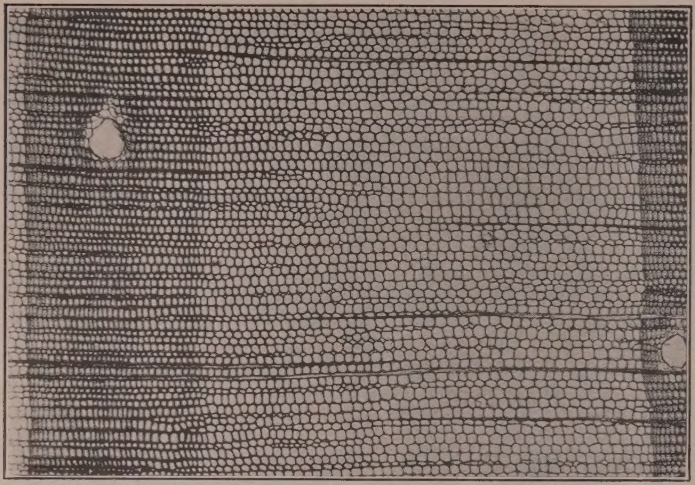


PLATE 2—PHOTOMICROGRAPH OF SHORTLEAF PINE

Note porous springwood and denser summerwood

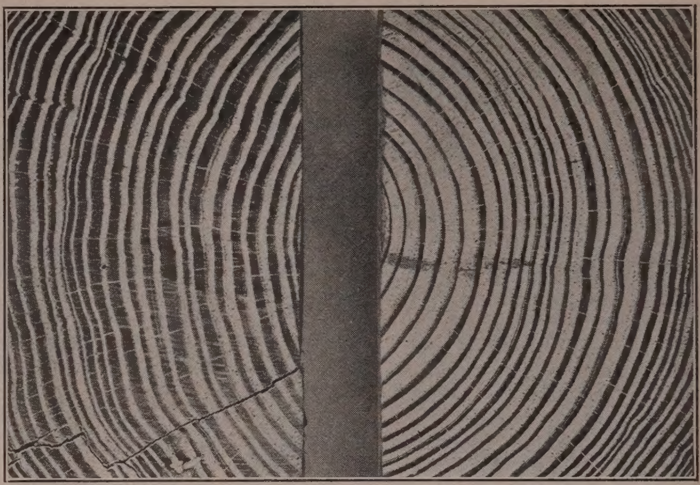


PLATE 3—CROSS-SECTIONS OF LOBLOLLY PINE BEAMS

Note different proportions of summerwood

resin appear to be very low, the presence of resin in wood keeps out moisture and air, and in this way it acts as a preservative. It is quite a common occurrence to find lying in the woods pine logs which at first sight are apparently in a fair state of preservation, but on examination the sap is found to be entirely decayed, the only portions remaining sound being such parts of the heart as are permeated with resin. It is also a well-known fact that fence posts chosen from pitchy material last longer than similar posts from wood with a small pitch content. It seems reasonable, therefore, to consider that a high per cent of resin is favorable to durability.

DIRECTION OF GRAIN

By grain is commonly meant the lines formed by cutting the rings of annual growth. If a stick is sawed from a log in such a way that these lines run diagonally across the stick instead of parallel to the edges the load that it will carry as a beam will be considerably reduced and, moreover, failure will be more complete when it occurs.

There is, however, another kind of cross-grain, known as spiral grain, which is more difficult to detect than that just described, but which is also liable to weaken the timber and cause complete failure. The wood of the conifers may be considered as built up mainly of elongated cells running lengthwise with the trunk, with a much smaller proportion of cells in small bundles located at right angles to them and lying radially. These groups or bundles of cells are termed pith rays. Their cross-sections appear on a magnified tangential or flat-grained face of timber as figures tapering on each end with the long axis vertical and on a radial section as light-colored bands. In oak, as an example, the pith rays are very much in evidence and form the figures in quartered material. In the conifers, however, the pith rays are not generally noticeable. Not infrequently the cells of which a tree is built up follow a spiral course around the pith instead of lying vertically. In such trees the cross-sections of the pith rays are also inclined or lie diagonally and form an indication of spiral grain. It is quite possible for a piece to be straight-grained, so far as the annual rings are concerned, and still to have a spiral grain. Spiral grain is often indicated in seasoned material by the diagonal direction of fine surface checks which occur at the pith rays. Splitting a block of wood will show whether or not it has spiral grain.

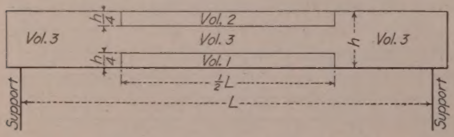


PLATE 4—DIVISION OF BEAM INTO VOLUMES FOR LOCATION OF KNOTS

MOISTURE

While the effect of moisture on strength is very marked in small clear pieces of wood (Plate 1), it is much less in evidence in large pieces. Under the present methods of seasoning structural material any increase in the strength of the wood fiber is commonly offset by the checks and shakes induced in the process, so that the strength of large beams on the average is increased little, if any, over that of the green sticks. The moisture condition of structural material from a durability standpoint is, however, an important consideration, especially in cases where the beams are to be placed in a poorly ventilated location. Under such conditions careful seasoning becomes necessary if even a reasonable life is to be expected, and it is highly probable that preservative treatment will be found desirable. Green timber is also liable to shrinkage or distortion.

PROPORTION OF SAPWOOD

The proportion of sapwood in a stick of timber has no bearing on its strength. If a tree is cut at a time when it is putting on its densest wood, pieces cut from the sapwood will be the stronger. If, on the other hand, it is cut when mature, so that the densest wood has changed to heartwood, then pieces cut from the heartwood will be the stronger. Sapwood is, however, notably less durable than heartwood, and generally only a small proportion of sapwood is allowed in the highest-class structural timber. If the timber is to be treated with a preservative, the presence of sapwood is not detrimental, as it is easier to treat than heartwood.

DEFECTS

Knots, checks, and shakes are the most common kinds of defects affecting strength. In timber for use as beams the influence of knots on strength is largely a matter of location. Plate 4 shows a method of dividing a beam into three volumes with reference to the location of knots. Numerous tests have shown that knots occurring in Volume 1, which occupies the lower quarter of the central half of the beam, have considerably more weakening effect than similar knots occurring in other volumes. Loose or rotten knots are, of course, more harmful than those closely knit with the surrounding wood. A comparatively small knot situated near enough the lower edge of a beam to turn the grain off is more harmful than a larger knot so placed as to allow the grain to be continuous in passing. In some cases knots near the neutral plane may act as pins and tend to strengthen a beam against failure in longitudinal shear. In a series of tests on loblolly pine beams those with knots in Volume 1 had about 75 per cent of the strength of sticks with knots in the remaining portions.

Checks are caused by the stresses set up in seasoning. Structural timber in large size is difficult to season without more or less checking, even under favorable conditions; frequently it is so exposed as to cause the surface to dry much more rapidly than moisture can be transmitted from the inner portions to the surface, and thus the outer portions dry and shrink, while the center is still wet, and checking results. A shake is a separation between two annual rings; generally the separation occurs in only part of the ring, but sometimes it is complete. Shakes are ascribed to the bending action of wind on the standing tree. They are frequently not visible in green timber, but show up later during seasoning. Both checks and shakes weaken beams in their ability to resist horizontal shear in proportion as they affect the area of the beam near the neutral axis.

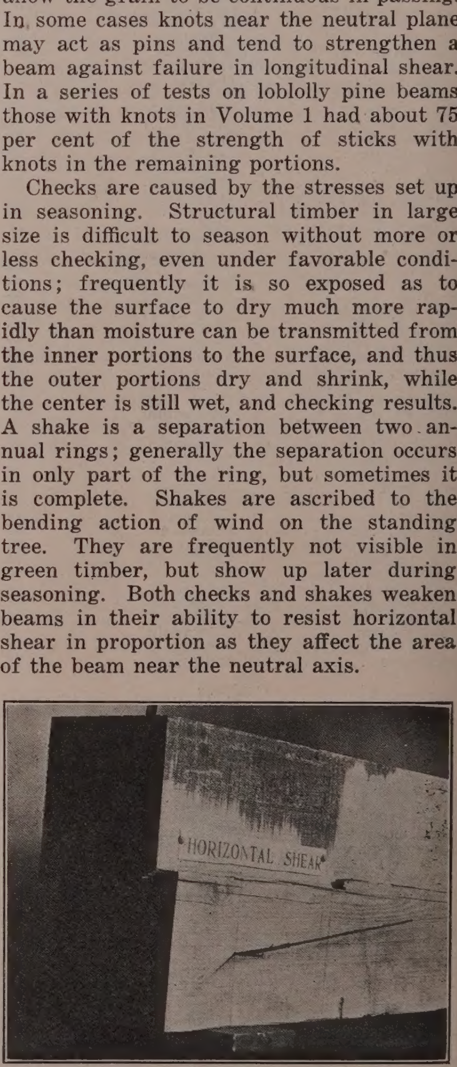
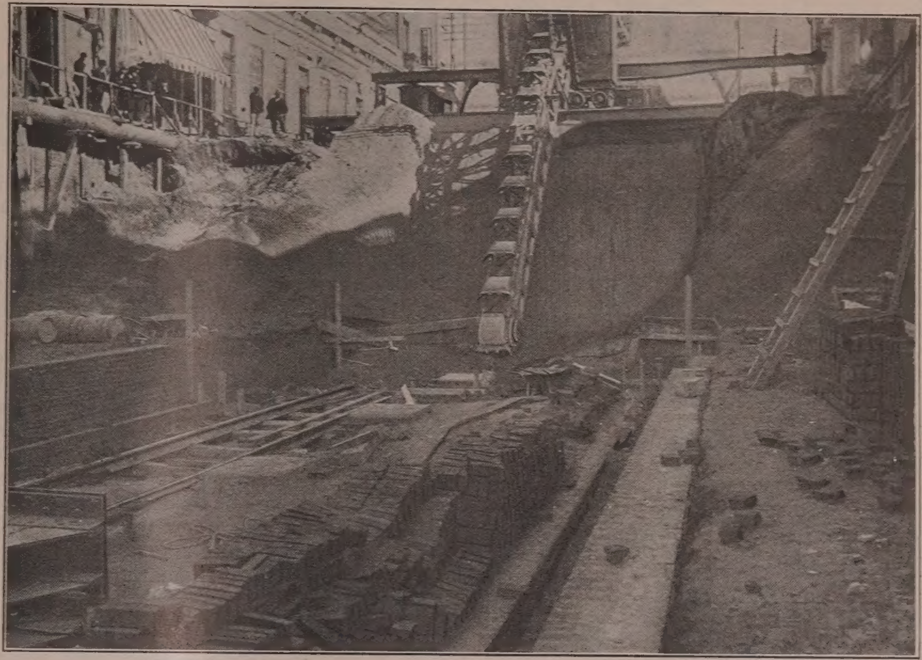


PLATE 5—HORIZONTAL SHEAR FAILURE DUE TO RING SHAKE

In the case of a longleaf pine stringer, 8 x 16 in. in section, tested in bending over a 15-ft. span, failure may occur by tension or tearing apart of the fibers in the lower part of the beam, by compression or crushing in the upper part, or by shearing along the neutral axis. The results of shearing tests on small clear pieces show that under the conditions given failure would occur in tension or compression long before the beam would have reached its maximum shearing stress, provided the shear resisting area was intact (Plate 5). In tests on commercial material, however, horizontal shear is a common form of failure due to the fact that the area that resists shear is frequently weakened by checks and shakes. A comparison of the stresses obtained from tests on 8 x 16 in. x 16-ft. stringers and small clear pieces shows that the shear-resisting area is frequently reduced about 50 per cent due to checks and shakes.

Checks and shakes influence durability of timber by allowing the collection of moisture and the formation of conditions suitable to the propagation of fungus growth. Knots have little effect on durability one way or the other, except in cases where they become loose and fall out, leaving holes that are apt to collect moisture and produce conditions favorable to decay.



EXCAVATING MACHINE WORKING ON TRANSVERSE FACE

Subway Construction at Buenos Aires

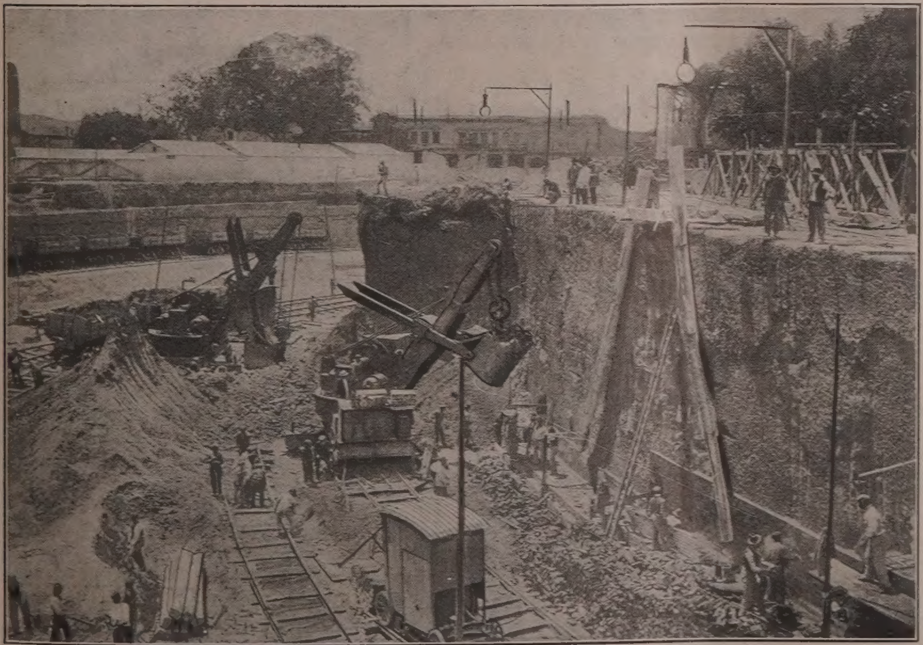
Deep Open Cut Excavation in Clay without Sheeting, Even near Buildings

A LONG section of double-track subway now under construction in Buenos Aires has been made in open cuts excavated to a depth of 15 to 30 ft. through very hard clay without any sheeting or retaining. Part of the excavation has been made by a ladder excavator working between preliminary side trenches. Some of the side walls are made of thin reinforced concrete slabs between vertical I-beams. The roof is made of transverse I-beams and premolded sectional concrete jack arches.

Several miles of open cut extension are now under construction for the Buenos Aires subway system described in the Engineering Record of Nov. 29, 1913, page 613. In one section long trenches were simultaneously opened on both sides of the street and in them the concrete side walls were built before the remainder of the excavation was completed. The hard clay encountered was so refractory that the vertical sides of the trench, sometimes 30 ft. or more in height, stood up without any sheeting or retaining, and four-story brick houses on the street lines were not underpinned and did not settle during the progress of the work.

EXCAVATING MACHINES

The soil between the side trenches was removed by an excavating machine of the ladder dredge type that spanned the full width of the excavation, traveling on two lines of rails on each side. The ladder was traversed back and forth on the bridge and its buckets excavated as they ascended the inclined surface of the face and dumped into a hopper by which the material was discharged to cars or wagons in the street. In the early part of the work steam shovels were used in the trench, but they have since been discontinued.



EARTH EXCAVATION WITH HIGH UNPROTECTED FACE

In some places the side walls have vertical I-beams with the panels between them filled by premolded concrete slabs 2 in. thick reinforced by wire rods. These blocks are inserted between the flanges of the I-beam columns. In most instances the walls are of monolithic concrete without reinforcement.

The transverse roof I-beams are carried on the concrete wall and on the wall columns, and the spaces between them are bridged by premolded concrete jack arches in sections 2 or 3 ft. long made without reinforcement. These arches are seated on the bottom flanges of the roofbeams and rise a little above the top flanges. The spaces between the arches are filled in to a level just above the crown with concrete forming the flat upper surface of the subway roof.

All of the concrete is made one part German Portland cement and seven parts coarse, sharp, clean, river sand mixed in Ransome machines. No stone is used and the concrete is really mortar.

The work is being executed by the Cia de Tramways Anglo-Argentina Limitada. Mr. Ruchs is chief engineer and Mr. Fels is assistant chief engineer.

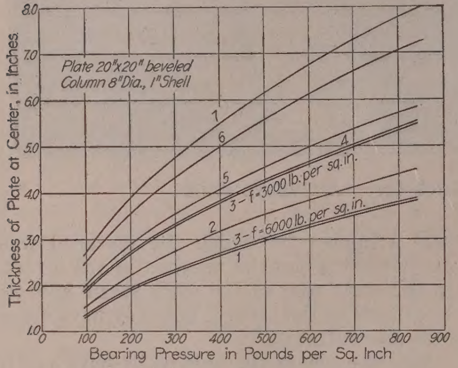
FIGHTING ICE at the St. Louis water-intake tower last winter cost $726, according to the last annual report of the water commissioner. The unusually open winter shortened the period to thirty-eight days, when extra men were required. On Feb. 14 ice gorges at Alton lowered the river stage to within 0.7 ft. of the lowest on record in 1909. No trouble was experienced in getting 120,000,000 gal. daily, even though ice was forming at the time, but the new south gate and an emergency gate were commissioned. An old cofferdam used in the original construction opposite the lowest gates was removed, and this increased the freedom of flow through the parts.

Letters to the Editor

Design of Cast-Iron Base Plates

SIR: I am submitting this information on base plates with the hope that some one else may have looked into the matter or have had some experience or data on the subject. It appears from the data available and the way in which most text books have ignored the matter that it has either been looked upon as unimportant or extremely simple. Nevertheless, as will be seen by the evidence given herewith, there is a big difference in the results from the various formulæ and it is doubtful which is the safest and at the same time most economical method to use.

The plotted curves readily show the difference in the seven following formulæ for a given size cast-iron base plate and column. In the formulæ, d = thickness at center of plate; d', thickness at edge of plate; l, projection from face of column to edge of plate; l', projection from inside of column to edge of plate; l'', one-half the width of plate; R, outside radius of column; r, inside radius of column; c, diameter of column; S, length of one side of plate; L'', perpendicular distance in inches from fracture line to center of downward pressure on one-half the area of plate; A', difference in area of plate and area of column; A'', area of column; K, allowable compressive strength of bearing material; W, total load delivered by column; @, per cent of resistance moment of a rectangular fracture section which is possessed by a tapered section with the same thickness d and the same value n, and f, fiber stress of material used for plate.



COMPARISON OF BASE-PLATE FORMULÆ

$$d = 0.01326l''\sqrt{K} \qquad (1)$$

$$d = l'\sqrt{K/2000} \qquad (2)$$

$$d = l\sqrt{3K/f} \qquad (3)$$

$$d = l''' \sqrt{2[(W - 2KA'')/A] \div f}$$

$$\text{where } l''' = (s/2) - (c/4) \qquad (4)$$

$$d = (\sqrt{KlA'/c}) \div 80 \qquad (5)$$

$$d = (l/50)\sqrt{6K\,[l + (c/2)]/(l + c)} \qquad (6)$$

$$d = \sqrt{100/@} \times (1/31.6)$$

$$\sqrt{(W/S)[(s/4) - L'']} \qquad (7)$$

It will be seen that there is a wide variation, too much so to allow it to pass unnoticed. Now look at the list of plates given below. The first three cases were tested to destruction. The remainder are from buildings from one to five years old. The latter will be found to come between the two extremes, but vary greatly.

Type of building	Size of plate, in.	d, in.	d', in.	Size of column, in.	Thickness of shell, in	Foundation material	Column load, lb.
Tests made by the University of Illinois	20 x 20	2 9/16	⅜	4	Solid	Folded blanket	155,000
	20 x 20	2 9/16	⅜	4	Solid	Wood cushion	268,000
	20 x 20	2 9/16	⅜	4	Solid	Wood cushion	213,500
Weave shed	20 x 20	2	1⅝	11 wood		Concrete	82,900
Storehouse	24 x 24	2¾	2⅜	11 x 11 wood		Concrete	85,250
Weave shed	16 x 16	1¾	1⅜	10 x 10 wood		Concrete	15,120
Mill	24 x 24	2½	2	8 dia.	⅞	Concrete	74,250
Storehouse	24 x 24	2½	2⅛	11½ dia., wood		Concrete	81,600
Mill	20 x 20	3	2⅝	11½ dia., wood		Brick	88,400
Manufact. building.	24 x 24	3	2⅝	9 dia.	⅞	Concrete	134,400
Manufact. building.	16 x 16	1¼	1¼	10 dia., wood		Brick	44,440

It might be well to call attention to the fact that most building laws call for an extreme fiber stress due to bending of from 2500 to 3000 lb. per square inch (tension side), whereas formulæ (1), (2) and (3) use a stress of 6000 lb. or thereabout.

To make a long story short, what should be considered a safe, economical design? The evidence has been presented in the foregoing. I will give no conclusions but hope to create enough interest in the subject to have others join in the discussion. A base plate is a small thing; 3 or 4 lb. of iron one way or the other in a single base plate amounts to nothing, but in a large building it forms an item worthy of consideration.

Cambridge, Mass. H. E. HOLLISTER.

Girderless Concrete Slabs

SIR: Referring to John Krippner's article on "Girderless Concrete Slabs" appearing in the Engineering Record of June 27, page 731, and his further comments on the subject in a letter in the issue of July 25, page 116, I should like to point out that while Mr. Krippner's method of analysis determines the correct bending moment around a ring called the column capital, there are two practical considerations that render his analysis of little value. In order to reinforce properly this ring around the column head for the bending moment found, the rods should all be radial. It is possible for only four of the rods in the four sets of bands to be radial. The others cannot pass through the center of the circle, many of them will lie far from the center and some of them will be entirely outside of the circle. Only 5 or 10 per cent of the rods may be radial, and thus in agreement with his assumption. The other practical consideration is that there is no real capital at the head of the column capable of supporting a load around this assumed circle. The support is only imaginary—it is the feather edge of an ornamental fillet on the column head.

When these features are correctly considered, and when it is further considered that the load is not always balanced over the column head, it can be shown that Mr. Krippner's formula for bending moments is from 100 to 200 per cent in error.

Pittsburgh, Pa. Edward Godfrey.

[Mr. Godfrey's letter was submitted to Mr. Krippner for comment. His reply follows.—EDITOR.]

SIR: In order to remove any erroneous impression as to what this method of computation can or cannot do for flat slabs, let me state again that it is intended to be used for uniformly distributed load only. Any special or unbalanced loading must receive due consideration and attention. For uniform load, however, this method gives coefficients which are perfectly on the safe side, and, if anything, they cannot be larger. Consider, for instance, a square panel supported in the middle parallel to the side. The bending moment will be $WL/8$ and the length of support, L (the length of panel). Now, the length of support in the method used in my article is the circumference of the assumed column capital. By making these lengths equal, or letting $\pi D = L$, it can be shown that the results are equal. It is necessary, however, to modify this equation somewhat, as the length of the elements producing the bending moment varies from $L - D$ to L sec45° $- D$, as will be seen on inspection. By taking the mean length half way between these extremes—i. e., for 22 deg. 30 min. — the equation will take this form:

$$\pi D = L \sec 22^\circ\, 30' - D,$$

$$\text{or } 3.14\, D = 1.08\, L - D,$$

from which $D = L/3.83$. For this value of the diameter it will be found in the table on page 732 of the Engineering Record of June 27 a coefficient of $WL/8$. Proceeding as before, this gives $WL/12$ and $WL/24$ for the support and for the middle respectively. This is the most that can be reasonably expected by this method, for in practice the column capital rarely exceeds two-ninths of the panel length. As, however, the slab around the head is usually thicker by 5 in. or more, this will tend to increase the column capital correspondingly, so that it may be reasonably expected to reach the above-mentioned value.

As to Mr. Godfrey's objection regarding the distribution of steel around the column head, it is shown by tests that the actual distribution of steel rods over the head is of little consequence if the rods are within reasonable distance from the center.

Concerning the column capital support, it must be evident that this enlargement of the column is put there to serve a useful purpose and not merely as an ornament. The sides are specified not to have a slope flatter than 45 deg. and the diameter is to be measured where the thickness of the sloping sides is not less than 1½ in. so as to avoid the feather-edge support.

All of this, however, is a matter of detail and is well covered in the ruling of this department (Department of Buildings). In view of the fact that the values recommended by the boards of examiners are $WL/15$ and $WL/25$ (which, by the way, is a valuable bit of information, as it cost over $25,000 to get), I think this method is a fair approximation to the results of tests when all the various elements entering into the problem are properly taken into account.

Chicago, Ill. JOHN KRIPPNER.

A RAILROAD-WRECK FATALIST has been disclosed in the person of a Justice of the Supreme Court of the State of Washington. A lower court had allowed an orphan damages for the death of his father, which was caused by a wreck that occurred March 1 at Wellington, on the Great Northern Railroad, when a snow slide carried two trains down the mountain side. In reversing this ruling the higher court stated that the wreck was caused by an act of God and that the railroad could not be held responsible.